2009 *NEHRP RECOMMENDED SEISMIC PROVISIONS FOR NEW BUILDINGS AND OTHER STRUCTURES*:

PART 2, COMMENTARY TO ASCE/SEI 7-05

This part of the 2009 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures presents commentary to ASCE/SEI 7-05 utilizing the chapter and section numbers of that standard. Commentary to the modifications of the standard that appear in Part 1 of this Provisions volume is presented at the end of each chapter of modifications and can be used to replace or add to this Part 2 Commentary (e.g., this Part 2 Commentary addresses the maps that appear in ASCE/SEI 7-05, not the new risk-targeted maps and procedures presented in Part 1 of this volume).

This commentary is intended primarily for design professionals and members of the codes- and standards-development community. However, an understanding of the basis for the seismic regulations contained in the nation's building codes and standards is important to many outside this technical community including elected officials and other decision makers responsible for aspects of the built environment, the financial and insurance communities, and individual business owners and other citizens. These individuals and others who do not have in-depth technical knowledge may find a complementary report that presents a brief overview of the 2009 Provisions of interest. This overview is published as FEMA P-749, Concepts of Earthquake-resistant Design: An Introduction to the NEHRP Recommended Seismic Provisions for New Buildings and Other Structures.

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COMMENTARY TO CHAPTER 11, SEISMIC DESIGN CRITERIA

C11.1 GENERAL

C11.1.1 Purpose. When prescribed wind loading governs the stress or drift design, the resisting system still must conform to the special requirements for seismic-force-resisting systems. This is required in order to resist, in a ductile manner, potential seismic loads in excess of the prescribed wind loads. A proper, continuous load path is an obvious design requirement, but experience has shown that it often is overlooked and that significant damage and collapse can result. The basis for this design requirement is two-fold:

1. To ensure that the design has fully identified the seismic-force-resisting system and its appropriate design level and
2. To ensure that the design basis is fully identified for the purpose of future modifications or changes in the structure.

Detailed requirements for analyzing and designing this load path are given in the appropriate design and materials chapters.

C11.1.2 Scope. The scope statement establishes in general terms the applicability of ASCE/SEI 7-05. Certain structures are exempt for the following reasons:

Exemption 1 – Detached one- and two-family dwellings in Seismic Design Categories A, B, and C, along with those located where $S_s < 0.4g$, are exempt because they represent low seismic risks.

Exemption 2 – Structures constructed using the conventional light-frame construction requirements in Section 12.5 are deemed capable of resisting the anticipated seismic forces. While specific elements of conventional light-frame construction may be calculated to be overstressed, typically there is a great deal of redundancy and uncounted resistance in such structures. Detached one- and two-story wood-frame dwellings generally have performed well even in regions of higher seismicity. Section 12.5 adequately provides the level of safety required for such dwellings without imposing any additional requirements.

Exemption 3 – Agricultural storage structures generally are exempt from most code requirements because of the exceptionally low risk to human life involved.

Exemption 4 – Bridges, transmission towers, nuclear reactors, and other structures with special configuration and uses are not covered. The regulations for buildings and building-like structures presented in this document do not adequately address the design and performance of such special structures.

ASCE/SEI 7-05 is not retroactive and usually applies to existing structures only when there is an addition, change of use, or alteration. Minimum acceptable seismic resistance of existing buildings is a policy issue normally set by the authority having jurisdiction. Appendix 11B of the standard contains rules of application for basic conditions. ASCE/SEI 31*, Seismic Evaluation of Buildings,* and ASCE/SEI 41, *Seismic Rehabilitation of Existing Buildings,* provide technical guidance but do not contain policy recommendations. A chapter in the *International Building Code* (IBC) applies to alteration, repair, addition, and change of occupancy of existing buildings, and the International Code Council maintains the *International Existing Building Code* (IEBC) and an associated commentary.

C11.1.4 Alternate Materials and Alternate Means and Methods of Construction. It is not possible for a design standard to provide criteria for the use of all possible materials and their combinations and methods of construction, either existing or anticipated. While not citing specific materials or methods of construction currently available that require approval, this section serves to emphasize that the evaluation and approval of alternate materials and methods require a recognized and accepted approval system. The requirements for materials and methods of construction contained within the document represent the judgment of the best use of the materials and methods based on well-established expertise and historical seismic performance. It is important that any replacement or substitute be evaluated with an understanding of all the ramifications of performance, strength, and durability implied by the standard.

It also is recognized that until needed standards and agencies are created, authorities having jurisdiction need to operate on the basis of the best evidence available to substantiate any application for alternates. If accepted standards are lacking, it is strongly recommended that applications be supported by extensive reliable data obtained from tests simulating, as closely as is practically feasible, the actual load and deformation conditions to which the material is expected to be subjected during the service life of the structure. These conditions, when applicable, should include several cycles of full reversals of loads and deformations in the inelastic range.

C11.4 SEISMIC GROUND MOTION VALUES[1]

The approach adopted in Section 11.4 is intended to provide for a uniform margin against collapse at the design ground motion. In order to accomplish this objective, ground motion hazards are defined in terms of maximum considered earthquake (MCE) ground motions, which are based on a set of rules that depend on the seismic hazard of a region. Design ground motions are based on a lower bound estimate of the margin against collapse inherent in structures designed to the seismic provisions in the standard. This lower bound was judged, based on experience, to correspond to a factor of about 1.5 in ground motion. Consequently, the design earthquake ground motion was selected at a ground shaking level that is 1/1.5 (or 2/3) of the MCE ground motion.

For most regions of the nation, the MCE ground motion is defined with a uniform probability of exceedance of 2 percent in 50 years (return period of about 2500 years). While stronger shaking than this could occur, it was judged that it would be economically impractical to design for such very rare ground motions and that the selection of the 2 percent probability of exceedance in 50 years as the MCE ground motion would result in acceptable levels of seismic safety.

In regions of high seismicity, such as in many areas of California, the seismic hazard is typically controlled by large-magnitude events occurring on a limited number of well-defined fault systems. Probabilistic ground motions calculated at a 2 percent probability of exceedance in 50 years can be much larger than deterministic ground motions computed based on the characteristic magnitudes of earthquakes on these known active faults. These probabilistic motions are greater if these major active faults produce characteristic earthquakes every few hundred years. For these regions, it is considered more appropriate to determine MCE ground motions directly by deterministic methods based on the characteristic earthquakes of these defined faults. In order to provide an appropriate level of conservatism in the design process when the deterministic approach is used to calculate MCE ground motion, the median ground motion estimated for the characteristic event is multiplied by 1.5.

C11.4.1 Mapped Acceleration Parameters. In the general procedure, these motions are computed from mapped values of the spectral response acceleration at short periods, S_S , and at 1 second, S_1 , for Class B sites. These S_s and S_1 values may be obtained directly from Figures 22-1 through 22-14 (in Chapter 22). Development of these maps is explained in detail in Appendix A of the *Part 2 – Commentary* volume of the 2003 *NEHRP Recommended Provisions*. The 2003 S_s and S_1 values also can be obtained from the U.S. Geological Survey (USGS) website: http://earthquake.usgs.gov/designmaps.

S_S is the mapped value of the 5-percent-damped MCE spectral response acceleration for short-period structures founded on Site Class B (firm rock) sites. The short-period acceleration has been determined at a period of 0.2 second because it was concluded that 0.2 second was reasonably representative of the shortest effective period of buildings and structures that are designed using the standard, considering the effects of soil compliance, foundation rocking, and other factors typically neglected in structural analysis.

Similarly, S_1 is the mapped value of the 5-percent-damped MCE spectral response acceleration at a period of 1 second on Site Class B. The spectral response acceleration at periods other than 1 second typically can be derived from the acceleration at 1 second. Consequently, for MCE ground shaking on Site Class B sites, these two response acceleration parameters, S_S and S_1, are sufficient to define an entire response spectrum for the period range of importance for most buildings and structures.

C11.4.3 and C11.4.4 Site Coefficients and Adjusted Acceleration Parameters. Using the general procedure to obtain acceleration response parameters that are appropriate for sites with a classification other than Site Class B, the S_S and S_1 values must be modified as indicated in Section 11.4.3. This modification is performed using two coefficients, F_a and F_v, that respectively scale the S_S and S_1 values determined for Site Class B to values appropriate for other site classes. The MCE spectral response accelerations adjusted for site class are designated S_{MS} and S_{M1}, respectively, for short-period and 1-second-period response. As described above, structural design in ASCE/SEI 7-05 is performed for earthquake demands that are 2/3 of the MCE response spectra. As set forth in Section 11.4.4, two additional parameters, S_{DS} and S_{D1}, are used to define the acceleration response spectrum for this design level event. These parameters are 2/3 of the respective S_{MS} and S_{M1} values and define a design response spectrum for sites of any characteristics and for natural periods of vibration less than the transition period, T_L. Values of S_{MS}, S_{M1}, S_{DS}, and S_{D1} can also be obtained from the USGS website cited above.

The site coefficients, F_a and F_v, presented respectively in Tables 11.4-1 and 11.4-2 for the various site classes are based on the results of empirical analyses of strong-motion data and analytical studies of site response.

The amount of ground-motion amplification by a soil deposit relative to bedrock depends on the wave-propagation characteristics of the soil, which can be estimated from measurements or inferences of shear-wave velocity and in the turn the

[1] Note that this section focuses on the methods and design procedures of ASCE/SEI 7-05 and the 2003 edition of the *Provisions*; commentary on the new risk-targeted maps and design procedures is presented in Part 1 of this volume following the modifications to ASCE 7 Section 11.4 and Chapter 22.

shear modulus for the materials as a function of the level of shaking. In general, softer soils with lower shear-wave velocities exhibit greater amplifications than stiffer soils with higher shear-wave velocities. Increased levels of ground shaking result in increased soil stress-strain nonlinearity and increased soil damping which, in general, reduces the amplification, especially for shorter periods. Furthermore, for soil deposits of sufficient thickness, soil amplification is generally greater at longer periods than at shorter periods.

An extensive discussion of the development of the F_a and F_v site coefficients is presented by Dobry, et al. (2000). Since the development of these coefficients and the development of a community consensus regarding their values in 1992, earthquake events have provided additional strong-motion data from which to infer site amplifications. Analyses conducted on the basis of these more recent data are reported by a number of researchers including Crouse and McGuire (1996), Dobry et al. (1999), Silva et al. (2000), Joyner and Boore (2000), Field (2000), Steidl (2000), Rodriquez-Marek et al. (2001), Borcherdt (2002), and Stewart et al. (2003). Although the results of these studies vary, the site amplification factors are generally consistent with those in Tables 11.4-1 and 11.4-2.

C11.4.5 Design Response Spectrum. The design response spectrum (Figure 11.4-1) consists of several segments. The constant-acceleration segment covers the period band from T_o to T_s; response accelerations in this band are constant and equal to S_{DS}. The constant-velocity segment covers the period band from T_s to T_L, and the response accelerations in this band are proportional to $1/T$ with the response acceleration at 1-sec period equal to S_{D1}. The long-period portion of the design response spectrum is defined on the basis of the parameter, T_L, the period that marks the transition from the constant-velocity segment to the constant-displacement segment of the design response spectrum. Response accelerations in the constant-displacement segment, where $T \geq T_L$, are proportional to $1/T^2$. Values of T_L are provided on maps in Figures 22-15 through 22-20.

The T_L maps were prepared following a two-step procedure. First, a correlation between earthquake magnitude and T_L was established. Then, the modal magnitude from deaggregation of the ground-motion seismic hazard at a 2-second period (1-second period for Hawaii) was mapped. Details of the procedure and the rational for it are found in Crouse et al. (2006).

C11.4.7 Site-Specific Ground Motion Procedures. The objective of a site-specific ground-motion analysis is to determine ground motions for local seismic and site conditions with higher confidence than is possible using the general procedure of Sections 11.4.

Near-source effects on horizontal response spectra for periods of vibration greater than approximately 0.5 second include directivity, which increases ground motions for fault rupture propagating toward the site, and directionality, which increases ground motions normal (perpendicular) to the strike of the fault. These effects are discussed in Somerville et al. (1997) and Abrahamson (2000).

C11.5 IMPORTANCE FACTOR AND OCCUPANCY CATEGORY

Large earthquakes are rare events that will include severe ground motions. Such events are expected to result in damage to structures even if they were designed and built in accordance with the minimum requirements of the standard. The consequence of structural damage or failure is not the same for the various types of structures located within a given community. Serious damage to certain classes of structures, such as critical facilities (e.g., hospitals), will disproportionally affect a community. The fundamental purpose of this section and subsequent requirements that depend on this section is to improve the ability of a community to recover from a damaging earthquake by tailoring the seismic protection requirements to the relative importance of that structure. That purpose is achieved by requiring better performance of those structures that:

1. Are necessary to response and recovery efforts immediately following an earthquake,
2. Present the potential for catastrophic loss in the event of an earthquake, or
3. House a very large number of occupants or occupants less able to care for themselves than the average.

The first basis for seismic design in the standard is that structures will have a suitably low likelihood of collapse in the very rare event defined as the maximum considered earthquake (MCE) ground motion. A second basis is that life threatening damage, primarily from failure of nonstructural elements in and on structures, will be unlikely in an unusual but less rare earthquake ground motion, which is given as the design earthquake ground motion (defined as two-thirds of the MCE). Given the occurrence of ground motion equivalent to the MCE, a population of structures built to meet these design objectives will probably still experience substantial damage in many structures, rendering these structures unfit for occupancy or use. Experience in past earthquakes around the world has demonstrated that there will be an immediate need to treat injured people, to extinguish fires and prevent conflagration, to rescue people from severely damaged or collapsed structures, and to provide sustenance to a population deprived of its normal means. Experience also has shown that these needs are best met when structures essential to response and recovery activities remain functional.

The standard addresses these objectives by requiring that each structure be assigned to one of the four occupancy categories presented in Chapter 1 and by assigning an importance factor to the structure based upon that occupancy category. (The two lowest categories, Ordinary and Low Hazard, are combined for all purposes within the seismic provisions). The occupancy category is then used as one of two components in determining the Seismic Design Category (see Section C11.6) and is a primary factor in setting drift limits for building structures under the design earthquake ground motion (see Section C12.12).

Figure C11.5-1 shows the combined intent of these requirements for design. The vertical scale is the likelihood of the ground motion with the MCE being the rarest considered. The horizontal scale is the level of performance of the structure and attached nonstructural components from collapse prevention at the low end to operational at the high end. (These performance levels are discussed further at other locations in the commentary.) The basic objective of collapse prevention at the MCE for ordinary structures (Occupancy Category II) is shown at the lower right by the solid triangle; protection from life-threatening damage at the design ground motion (defined by the standard as two-thirds of the MCE) is shown by the open triangle. The performance implied for higher occupancy categories is shown by square and circles. The performance anticipated for less severe ground motion is shown by dotted symbols. The three (net) classes and the numerical values assigned are far too coarse to assure the portrayed outcome for all structures, but it is judged to be adequate for the purpose given present limitations of knowledge and tools.

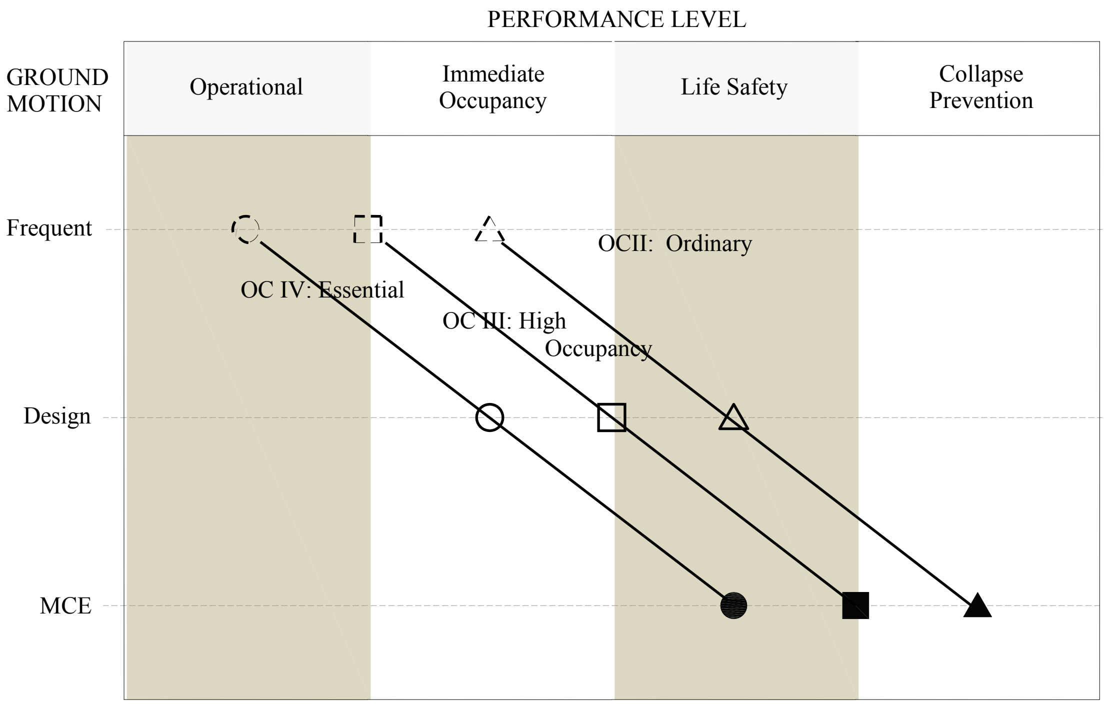

Figure C11.5-1 Expected performance as related to occupancy category (OC) and level of ground motion.

C11.5.1 Importance Factor. The importance factor is used throughout the standard in quantitative criteria for strength. In most of those quantitative criteria, the importance factor is shown as a divisor on the factor R or R_p in order to send a message to designers that the objective is to reduce damage for important structures in addition to preventing collapse in larger ground motions. The R and R_p factors adjust the computed linear elastic response to a value appropriate for design; in many structures, the largest component of that adjustment is ductility (the ability of the structure to undergo repeated cycles of inelastic strain in opposing directions). Inelastic strain damages a structure so, for a given strength demand, reducing the effective R factor (by means of the importance factor) increases the required yield strength, thus reducing ductility demand and related damage.

C11.5.2 Protected Access for Category IV Structures. Those structures considered essential facilities for response and recovery efforts must be accessible to carry out their purpose. For example, if the collapse of a simple canopy at a hospital could block ambulances from the emergency room admittance area, the canopy must meet the same structural standard as the hospital. This requirement must be considered in the siting of essential facilities in densely built urban areas.

C11.6 SEISMIC DESIGN CATEGORIES

Seismic design categories (SDCs) provide a means to step progressively from simple, easily performed design and construction procedures and minimums to more sophisticated, detailed, and costly requirements as both the level of seismic hazard and the consequence of failure escalate. The SDCs are used to trigger requirements that are not scalable; such requirements are either on or off. For example, the basic amplitude of ground motion for design is scalable – the quantity simply increases in a continuous fashion as one moves from a low hazard area to a high hazard area. However, a requirement to avoid weak stories is not particularly scalable. Requirements such as this create step functions. There are many such requirements in the standard, and the SDCs are used systematically to group these step functions. (Further examples include whether seismic anchorage of nonstructural items is required or not, whether particular inspections will be required or not, and height limits applied to various structural systems.)

In this regard, SDCs perform one of the functions of the seismic zones used in earlier U.S. building codes and still in use throughout much of the world. However, SDCs also are dependent on a building's occupancy and, therefore, its desired performance. Further, unlike the traditional implementation of seismic zones, the ground motions used to define the SDCs include the effects of individual site conditions on probable ground-shaking intensity.

In developing the ground-shaking limits for the various Seismic Design Categories and the design requirements for each, the equivalent modified Mercalli intensity (MMI) of various shaking spectra were considered. There are now various correlations of the qualitative MMI with quantitative characterizations of ground. The reader is encouraged to consult any of a great many sources that describe the MMIs. The following list is a very coarse generalization:

MMI V	No real damage
MMI VI	Light nonstructural damage
MMI VII	Hazardous nonstructural damage
MMI VIII	Hazardous damage to susceptible structures
MMI IX	Hazardous damage to robust structures

When the current design philosophy was adopted (the 1997 edition of the *NEHRP Recommended Provisions*, FEMA 302, and *Commentary*, FEMA 303), the upper limit for SDC A was set at roughly one-half of the lower threshold for MMI VII and the lower limit for SDC D was set at roughly the lower threshold for MMI VIII. However, the lower limit for SDC D was more consciously established by equating that design value (two-thirds of the MCE) to one-half of what had been the maximum design value in building codes over the period of 1975 to 1995. As more correlations between MMI and numerical representations of ground motion have been created, it is reasonable to make the following correlation between the MMI at MCE ground motion and the Seismic Design Category (all this discussion is for ordinary occupancies):

MMI V	SDC A
MMI VI	SDC B
MMI VII	SDC C
MMI VIII	SDC D
MMI IX	SDC E

An important change was made to the determination of SDC when the current design philosophy was adopted. Earlier editions of the *Provisions* utilized the peak velocity-related acceleration, A_v, to determine a building's Seismic Performance Category. However, this coefficient does not adequately represent the damage potential of earthquakes on sites with soil conditions other than rock. Consequently, the 1997 *Provisions* adopted the use of response spectral acceleration parameters S_{DS} and S_{D1}, which include site soil effects for this purpose.

Except for the lowest level of hazard (SDC A), the SDC also depends on the occupancy categories. For a given level of ground motion, the SDC is one category higher for Occupancy Category IV structures than for lower-risk structures. This has the effect of increasing the confidence that the design and construction requirements will deliver the intended performance in the extreme event.

Note that the tables in the standard are at the design level, defined as two-thirds of the MCE level. Also recall that the MMIs are qualitative by their nature and that the above correlation will be more or less valid depending on which numerical correlation for MMI is used. The numerical correlations for MMI roughly double with each step so correlation between design earthquake ground motion and MMI is not as simple or convenient.

In sum, at the MCE level, SDC A structures should not see motions that are normally destructive to structural systems, whereas the MCE level motions for SDC D structures can destroy vulnerable structures. The grouping of step function requirements by SDC is such that there are a few basic structural integrity requirements imposed at SDC A graduating to a suite of requirements at SDC D based upon observed performance in past earthquakes, analysis, and laboratory research.

The nature of ground motions within a few kilometers of a fault can be very different from more distant motions. For example, some near fault motions will have strong velocity pulses, associated with forward rupture directivity, that tend to be highly destructive to irregular structures even if they are well detailed. For ordinary occupancies, the boundary between SDCs D and E is set to define sites likely to be close enough to a fault that these unusual ground motions may be present. Note that this boundary is defined in terms of mapped bedrock outcrop motions affecting response at 1 second, not site adjusted values, in order to better discriminate between sites near and far from faults. Short-period response is not normally as affected as the longer period response. The additional design criteria imposed on structures in SDCs E and F specifically are intended to provide acceptable performance under these very intense near-fault ground motions.

For most buildings, the SDC is determined without consideration of the building's period. Structures are assigned to a SDC based on the more severe condition determined from 1-second acceleration and short-period acceleration. This is done for several reasons. Perhaps the most important of these is that it is often difficult to estimate precisely the period of a structure using default procedures contained in the standard. Consider, for example, the case of rigid wall/flexible diaphragm buildings including low-rise reinforced masonry and concrete tilt-up buildings with either untopped metal deck or wood diaphragms. The formula in the standard for determining the period of vibration of such buildings is based solely on the height of the structure and the length of wall present. These formulas typically indicate very short periods for such structures, often on the order of 0.2 second or less. However, the actual dynamic behavior of these buildings often is dominated by the flexibility of the diaphragm – a factor neglected by the approximate period formula. Large buildings of this type can have actual periods on the order of 1 second or more. In order to avoid misclassifying a building's SDC by inaccurately estimating the structural period, the standard generally requires that the more severe SDC determined on the basis of short- and long-period shaking be used.

Another reason for this requirement is a desire to simplify building regulation by requiring all buildings on a given soil profile in a particular region to be assigned to the same SDC regardless of the structural type. This has the advantage of permitting uniform regulation of structural system selection, inspection and testing requirements, seismic design requirements for nonstructural components, and similar aspects of the design process regulated on the basis of SDC, within a community.

Notwithstanding the above, it is recognized that classification of a building as SDC C instead of B or D can have significant impact on the cost of construction. Therefore, the 2005 edition of the standard includes an exception permitting the classification of buildings that can reliably be classified as having short structural periods on the basis of short-period shaking alone.

Local or regional jurisdictions enforcing building regulations may desire to consider the effect of the maps, typical soil conditions, and Seismic Design Categories on the practices in their jurisdictional areas. For reasons of uniformity of practice or reduction of potential errors, adopting ordinances could stipulate particular values of ground motion, particular site classes, or particular Seismic Design Categories for all or part of the area of their jurisdiction. For example:

1. An area with a historical practice of high seismic zone detailing might mandate a minimum SDC of D regardless of ground motion or site class.
2. A jurisdiction with low variation in ground motion across the area might stipulate particular values of ground motion rather than requiring use of the maps.
3. An area with unusual soils might require use of a particular Site Class unless a geotechnical investigation proves a better Site Class.

C11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

Seismic Design Category A is assigned when the MCE ground motions are well known to be below those normally associated with hazardous damage. Damaging earthquakes are not unknown or impossible in such regions, however, and ground motions close to such events may be large enough to produce serious damage. Providing a minimum level of resistance reduces both the radius over which the ground motion exceeds structural capacities and resulting damage in such rare events. There are reasons beyond seismic risk for minimum levels of structural integrity.

The requirements for SDC A are all minimum strengths for structural elements stated as forces at the level appropriate for direct use in the strength design load combinations. The two fundamental requirements are a minimum strength for a structural system to resist lateral forces and a minimum strength for connections of structural members.

For many buildings the wind force will control the strength of the lateral-force-resisting system but, for low-rise buildings of heavy construction with large plan aspect ratio, the minimum lateral force specified here may control. Note that the requirement is for strength and not for toughness, energy dissipation capacity, or some measure of ductility. The force level is not tied to any postulated seismic ground motion. The boundary between SDCs A and B is based on a spectral response acceleration of 25 percent of gravity (MCE level) for short-period structures; clearly the 1 percent acceleration level (Equation 11.7-1) is far smaller. For ground motions below the A/B boundary, the spectral displacements generally are on the order of a few inches or less depending on period. Experience has shown that even a minimal strength is beneficial in providing resistance to small ground motions, and it is an easy provision to implement in design. The low probability of motions greater than the MCE is a factor in taking the simple approach without requiring details that would produce a ductile response. Another factor is that larger design forces are specified for connections between main elements of the lateral force load path.

The minimum connection force is specified in three ways: a general minimum horizontal capacity for all connections; a special minimum for horizontal restraint of beams and trusses in line, which also includes the live load on the member; and a special minimum for horizontal restraint of concrete and masonry walls perpendicular to their plane. The 5 percent coefficient used for the first two is a simple and convenient value that provides some margin over the minimum strength of the system as a whole. The value for anchorage of concrete and masonry walls is simply scaled upward from the value of 200 pounds per linear foot traditionally used in past building codes for allowable stress design.

C11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

In addition to this commentary, Part 3 of the 2009 *NEHRP Recommend Provisions* includes additional and more detailed discussion and guidance on evaluation of geologic hazards and determination of seismic lateral pressures.

C11.8.1 Site Limitation for Seismic Design Categories E and F. Because of the difficulty of designing a structure for the direct shearing displacement of fault rupture and the relatively high seismic activity of SDCs E and F, locating a structure on an active fault having the potential to cause rupture of the ground surface at the structure is prohibited.

C11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F. The dynamic lateral earth pressure on basement and retaining walls during earthquake ground shaking is considered to be an earthquake load, E, for use in design load combinations. This dynamic earth pressure is superimposed on the pre-existing static lateral earth pressure during ground shaking. The pre-existing static lateral earth pressure is considered to be an H load.

Liquefaction potential should be evaluated for design earthquake ground motions consistent with peak ground accelerations of $S_{DS}/2.5$. The occurrence and consequences of geologic hazards for MCE ground motions also should be considered when evaluating structural stability and other pertinent performance criteria.

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COMMENTARY TO CHAPTER 12, SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

C12.1 STRUCTURAL DESIGN BASIS

The performance expectations for structures designed in accordance with ASCE/SEI 7-05 are described in Sections C11.1 and C11.5. Structures designed in accordance with the standard are likely to have a low probability of collapse but suffer serious damage if subjected to the maximum considered earthquake (MCE) or stronger ground motion. The uncertainty in performance results from variability of both ground motion and structural characteristics.

Earthquakes load structures indirectly. As the ground displaces, a structure follows and vibrates. The vibration produces structural deformations with associated strains and stresses. Computation of dynamic response to earthquake ground shaking is complex. The basic methods of analysis in the standard employ the common simplification of a response spectrum. A response spectrum for a specific earthquake ground motion approximates the maximum value of response to that ground motion for simple structures without reflecting the total time history of response. The design response spectrum specified in Section 11.4 and used in the basic methods of analysis in Chapter 12 is a smoothed and normalized approximation for many different ground motions.

Although the seismic requirements of the standard are stated in terms of forces and loads, there are no external forces applied to the above-ground portion of a structure during an earthquake. The design forces are intended only as approximations to generate internal forces suitable for proportioning the strength of structural elements and for estimating the deformations (when multiplied by the deflection amplification factor, C_d) that would occur in the same structure in the event of design-level (not MCE) ground motion.

C12.1.1 Basic Requirements. Chapter 12 of the standard sets forth a set of coordinated requirements that must be used together. The basic steps in structural design for acceptable seismic resistance are as follows:

1. Select gravity- and seismic-force-resisting systems appropriate to the anticipated intensity of ground shaking. Section 12.2 sets forth limitations depending on the Seismic Design Category.
2. Lay out these systems to produce a continuous, regular, and redundant load path so that the structures act as integral units in responding to ground shaking. Section 12.3 addresses configuration and redundancy issues.
3. Analyze a mathematical model of the structure subjected to lateral seismic motions and gravity forces. Sections 12.6 and 12.7 set forth requirements for the method of analysis and for construction of the mathematical model.
4. Proportion members and connections to have adequate lateral and vertical strength and stiffness. Section 12.4 specifies how the effects of gravity and seismic loads are to be combined to establish required strengths, and Section 12.12 specifies deformation limits for buildings.

One- to three-story structures with shear wall or braced frame systems of simple configuration may be eligible for design under the simplified alternative contained in Section 12.14. Any other deviations from the requirements of Chapter 12 are subject to approval and must be rigorously consistent as specified in Section 11.1.4.

The baseline seismic forces for proportioning structural elements (individual members, connections, and supports) are static horizontal forces derived from a linear elastic response spectrum procedure. A basic requirement is that horizontal motion can come from any direction, with detailed requirements being provided in Section 12.5. For most structures, the effect of vertical ground motions is not analyzed specifically; it is included in an approximate fashion by adjusting the load factors for dead load up and down, as described in Section 12.4. Certain conditions requiring more detailed analysis of vertical response are defined in Chapters 13 and 15 for nonstructural components and nonbuilding structures, respectively.

Higher levels of seismic analysis are permitted (and encouraged) for any structure and are required for some structures (see Section 12.6), but lower limits based on the equivalent lateral force procedures apply. The basic procedure uses response spectra that are representative of, but substantially reduced from, the anticipated ground motions. As a result, at the MCE level of ground shaking, structural elements are expected to yield, buckle, or otherwise behave inelastically.

This approach has substantial historical precedent. In past earthquakes, structures with appropriately ductile, regular, continuous systems designed for reduced forces have performed acceptably. In the standard, such design forces are computed by dividing the forces that would be generated in a structure behaving linearly when subjected to the design ground motion by the response modification coefficient, R, and the design ground motion is taken as two-thirds of the MCE ground motion.

The elastic deformations calculated under these reduced design forces are multiplied by the deflection amplification factor, C_d, to estimate the deformations likely to result from the design ground motion. As set forth in Sections 12.12 and 13, the amplified deformations are used to assess story drifts and to determine seismic demands on elements of the structure that are not part of the seismic-force-resisting system and on nonstructural components within structures. Where C_d is substantially less than R, the system is considered to have damping greater than the nominal 5 percent of critical damping.

The seismic-force-resisting system is expected to reach significant yield for forces in excess of the design forces. Significant yield is the point where complete plastification of the most critical region of the structure (e.g., formation of a first plastic hinge in the structure) occurs, not the point where first yield occurs in any member. Figure C12.1-1 shows the lateral force versus deformation relation for a typical structure. Significant yield is shown as the lowest yield hinge on the force-deformation diagram. With increased lateral loading, additional plastic hinges form and the resistance increases (following the solid curve) until a maximum is reached. The maximum resistance developed along the curve is substantially higher than that at first significant yield, and the margin is referred to as the overstrength capacity.

The provisions of the standard contemplate a seismic-force-resisting system with redundant characteristics wherein significant structural overstrength above the level of significant yield can be obtained by plastification at other points in the structure prior to the formation of a complete mechanism. The overstrength obtained by this continued inelastic action provides the reserve strength necessary for the structure to resist the extreme motions of the actual seismic forces that may be generated by the design ground motion.

The structural overstrength described above results from the development of sequential plastic hinging in a properly designed, redundant structure. Several other sources will further increase structural overstrength. First, material overstrength (i.e., actual material strengths higher than the nominal material strengths specified in the design) may increase the structural overstrength significantly. For example, a recent survey shows that the mean yield strength of A36 steel is about 30 to 40 percent higher than the minimum specified strength used in design calculations. Second, member design strengths usually incorporate a strength reduction (or resistance) factor, Φ, to produce a low probability of failure under design loading. Third, designers themselves introduce additional overstrength by selecting sections or specifying reinforcing patterns that exceed those required by the computations. Similar situations occur where prescriptive minimums of the standard, or of the design standards referenced from it, control the design. Finally, the design of many flexible structural systems (e.g., moment resisting frames) often is controlled by the drift rather than strength limitations of the standard with sections selected to control lateral deformations rather than to provide the specified strength.

The result is that structures typically have a much higher lateral resistance than that specified as a minimum by the standard, and first significant yielding of structures may occur at lateral load levels that are 30 to 100 percent higher than the prescribed design seismic forces. If provided with adequate ductile detailing, redundancy and regularity, full yielding of structures may occur at load levels that are two to four times the prescribed design force levels.

Most structural systems have some components or limit states that cannot provide reliable inelastic response or energy dissipation. Such components or limit states must be designed considering that the actual forces in the structure will be larger than those at first significant yield. The standard specifies an overstrength factor, Ω_0, to amplify the prescribed forces for use in design of such components or limit states. This specified overstrength factor is neither an upper nor a lower bound; it is simply an approximation specified to provide a nominal degree of protection against undesirable behavior.

Figure C12.1-1 illustrates the significance of design parameters contained in the standard including the response modification coefficient, R; the deflection amplification factor, C_d; and the system overstrength factor, Ω_0. These design values, provided in Table 12.2-1, as well as the criteria for story drift and P-delta effects, have been established considering the characteristics of typical properly designed structures. The actual structural overstrength, Ω, often will be less than the tabulated factor, Ω_0. This means that the required ductility, R_d, usually will exceed R/Ω_0. If excessive "optimization" of a structural design is performed with lateral resistance provided by only a few elements, the successive yield hinge behavior depicted in Figure C12.1-1 will not be able to form, the actual overstrength (Ω) will be small, and use of the design parameters in the standard may not provide the intended seismic performance.

The response modification coefficient, R, represents the ratio of the forces that would develop under the specified ground motion if the structure had entirely linear-elastic response to the prescribed design forces (see Figure C12.1-1). The structure must be designed so that the level of significant yield exceeds the prescribed design force. The ratio R, expressed as $R = V_E / V_S$, is always larger than 1.0; thus, all structures are designed for forces smaller than those the design ground motion would produce in a structure with completely linear-elastic response. This reduction is possible for a number of reasons. As the structure begins to yield and deform inelastically, the effective period of response of the structure lengthens which, for most structures, results in a reduction in strength demand. Furthermore, the inelastic action results in a significant amount of energy dissipation (hysteretic damping) in addition to other sources of damping present below significant yield. The combined effect, which is also known as the ductility reduction, explains why a properly designed structure with a fully yielded strength (V_y in Figure C12.1-1) that is significantly lower than the elastic seismic force demand (V_E in Figure C12.1-1) can be capable of providing satisfactory performance under the design ground motion excitations.

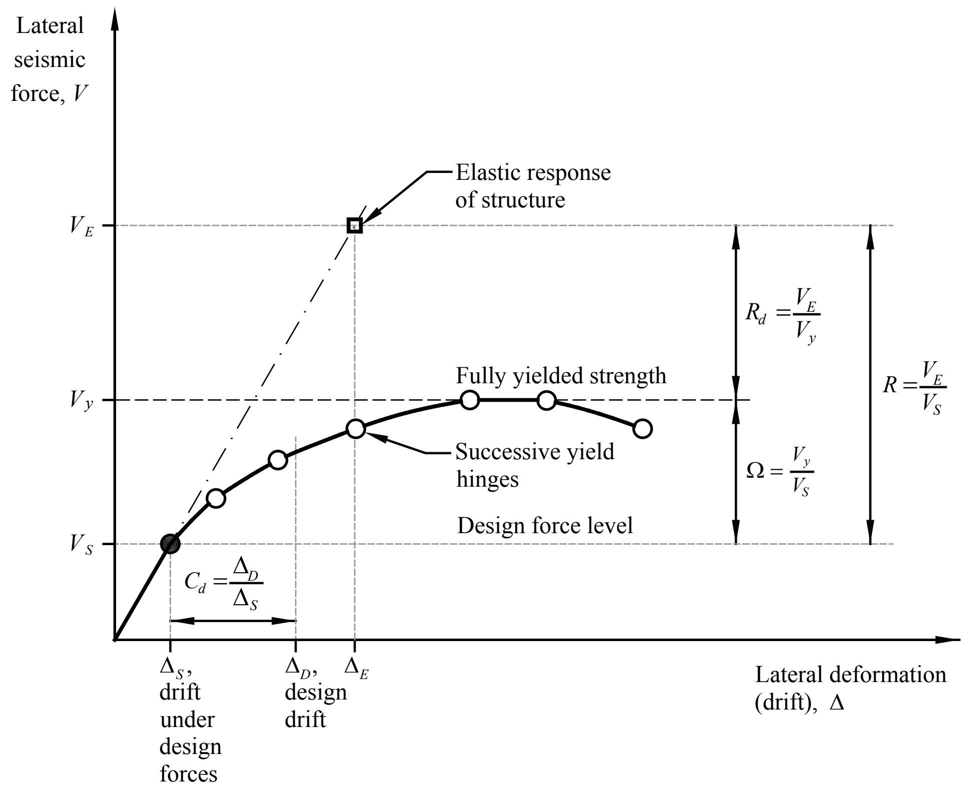
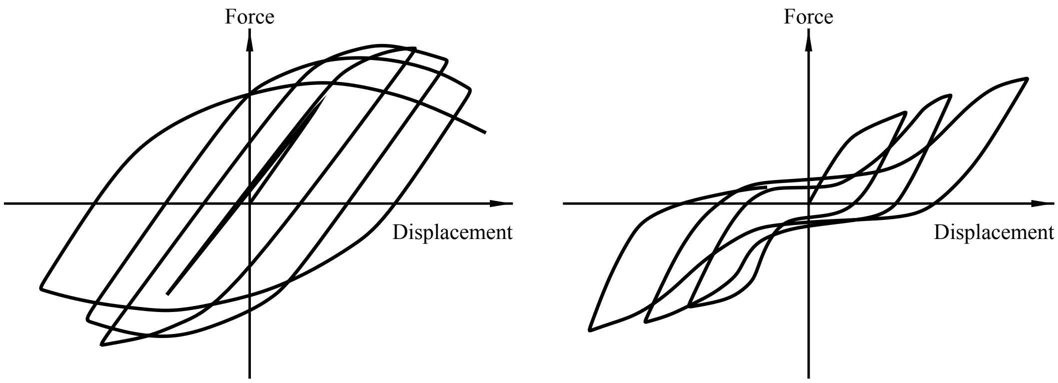


Figure C12.1-1 Inelastic force-deformation curve.

The energy dissipation resulting from hysteretic behavior can be measured as the area enclosed by the force-deformation curve of the structure as it experiences several cycles of excitation. Some structures have far more energy dissipation capacity than others. The extent of energy dissipation capacity available depends largely on the amount of stiffness and strength degradation the structure undergoes as it experiences repeated cycles of inelastic deformation. Figure C12.1-2 shows representative load deformation curves for two simple substructures such as a beam-column assembly in a frame. Hysteretic curve (a) in the figure is representative of the behavior of substructures that have been detailed for ductile behavior. The substructure can maintain nearly all of its strength and stiffness over several large cycles of inelastic deformation. The resulting force-deformation "loops" are quite wide and open, resulting in a large amount of energy dissipation. Hysteretic curve (b) represents the behavior of a substructure that has not been detailed for ductile behavior. It loses stiffness rapidly under inelastic deformation, and the resulting hysteretic loops are quite pinched. Such a substructure has much less energy dissipation than that for the substructure (a) but has a greater change in response period. The structural response is determined by a combination of energy dissipation and period modification.

The R values in the standard are based largely on engineering judgment of the performance of the various materials and systems in past earthquakes. The R factor for a specific project should be chosen and used with care. For example, lower values should be used for structures possessing a low degree of redundancy wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength. This situation can result in considerably more detrimental P-delta effects. Since it is difficult for individual designers to judge the extent to which R factors should be adjusted based on the inherent redundancy of their designs, Section 12.3.4 provides a coefficient, ρ, that is calculated based on the removal of individual seismic-force-resisting elements.

C12.1.2 Member Design, Connection Design, and Deformation Limit. Given that key elements of the seismic-force-resisting system will likely yield in response to ground motions as discussed in Section C12.1.1, it might be expected that structural connections would be required to develop the strength of connected members. Although that is a logical procedure, it is not a general requirement. The actual requirement varies by system and generally is specified in the standards for design of the various structural materials cited by reference in Section 14. Good seismic design requires careful consideration of this issue.



a. Ductile hysteresis loops

b. Pinched hysteresis loops

Figure C12.1-2 Typical hysteretic curves.

C12.1.3 Continuous Load Path and Interconnection. In effect, Section 12.1.3 calls for the seismic design to be complete and in accordance with the principles of structural mechanics. The loads must be transferred rationally from their point of origin to the final point of resistance. This should be obvious, but it often is overlooked by those inexperienced in earthquake engineering. Design consideration should be given to potentially adverse effects where there is a lack of redundancy. Given the many unknowns and uncertainties in the magnitude and characteristics of earthquake loading, in the materials and systems of construction for resisting earthquake loadings and in the methods of analysis, good earthquake engineering practice has been to provide as much redundancy as possible in the seismic-force-resisting system of buildings. Redundancy plays an important role in determining the ability of the building to resist earthquake forces. In a structural system without redundant components, every component must remain operative to preserve the integrity of the building structure. On the other hand, in a highly redundant system, one or more redundant components may fail and still leave a structural system that retains its integrity and can continue to resist lateral forces, albeit with diminished effectiveness.

While a redundancy requirement is included in Section 12.3.4, overall system redundancy can be improved by making all joints of the vertical load-carrying frame moment resisting and incorporating them into the seismic-force-resisting system. These multiple points of resistance can prevent a catastrophic collapse due to distress or failure of a member or joint. (The overstrength characteristics of this type of frame are discussed in Section C12.1.1.) The designer should be particularly aware of the proper selection of *R* when using only one- or two-bay rigid frames in one direction for resisting seismic loads. A single, one-bay frame or a pair of such frames provides little redundancy so the designer may wish to consider a reduced *R* to account for a lack of redundancy if the calculated redundancy is considered to be too low. As more one-bay frames are added to the system, however, overall system redundancy increases. The increase in redundancy is a function of frame placement and total number of frames.

The minimum connection forces are not intended to be applied simultaneously to the entire seismic-force-resisting system.

C12.1.4 Connection to Supports. The requirement is the same as given in Section 11.7.4 for Seismic Design Category A. See Section C11.7.

C12.1.5 Foundation Design. Most foundation design criteria are still stated in terms of allowable stresses, and the forces computed in the standard are all based on the strength level of response. When developing strength-based criteria for foundations, all the factors cited in Section 12.1.5 require careful consideration. Section C12.13 provides specific guidance.

C12.1.6 Material Design and Detailing Requirements. The design limit state for resistance to an earthquake is unlike that for any other load within the scope of the standard. The earthquake limit state is based on overall system performance, not member performance, where repeated cycles of inelastic straining are accepted as an energy dissipating mechanism. Provisions that modify customary requirements for proportioning and detailing structural members and systems are provided to produce the desired performance.

C12.2 STRUCTURAL SYSTEM SELECTION

C12.2.1 Selection and Limitations. For purposes of these seismic analyses and design requirements, seismic-force-resisting systems are grouped into categories as shown in Table 12.2-1. These categories are subdivided further for various types of vertical elements used to resist seismic forces. In addition, the sections for detailing requirements are specified.

Specification of R factors requires considerable judgment based on knowledge of actual earthquake performance as well as research studies. The factors in Table 12.2-1 continue to be reviewed in light of recent research results. R values for the various systems were selected considering observed performance during past earthquakes, the toughness (ability to dissipate energy without serious degradation) of the system, and the amount of damping typically present in the system when it undergoes inelastic response. FEMA P-695, *Quantification of Building Seismic Performance Factors* (Applied Technology Council, 2009) has been developed with the purpose of establishing and documenting a methodology for quantifying building system performance and response parameters for use in seismic design. While the response modification coefficient (R factor) is a key parameter being addressed, related design parameters such as the system overstrength factor (Ω_0) and deflection amplification factor (C_d) also are addressed. Collectively, these terms are referred to as "Seismic Performance Factors" (SPFs). Future systems will likely derive their SPFs using this methodology and existing system SPFs also may be reviewed in light of this new procedure.

Building height limits have been specified in codes and standards for over 50 years. The structural system limitations and building height limits specified in Table 12.2-1 evolved from these initial limitations and were further modified by the collective expert judgment of the PUC and the ATC-3 project team (the forerunners of the PUC). They have continued to evolve over the past 30 years based on observations and testing, but the specific values are based on subjective judgment.

In a bearing wall system, major load-carrying columns are omitted and the walls carry a major portion of the gravity (dead and live) loads. The walls supply in-plane lateral stiffness and strength to resist wind and earthquake loads as well as other lateral loads. In some cases, vertical trusses are employed to augment lateral stiffness. In general, this system has comparably lower values of R than other systems due to the frequent lack of redundancy for support of vertical and horizontal loads.

In a building frame system, gravity loads are carried primarily by a frame supported on columns rather than by bearing walls. Some portions of the gravity load may be carried on bearing walls, but the amount carried should represent a relatively small percentage of the floor or roof area. Lateral resistance is provided by shear walls or braced frames. Light-framed walls with shear panels are intended for use only with wood and steel building frames. Although gravity-load-resisting systems are not required to provide lateral resistance, most of them do. To the extent that the gravity-load-resisting system provides additional lateral resistance, it will enhance the building's seismic performance capability, so long as it is capable of resisting the resulting stresses and undergoing the associated deformations.

In a moment-resisting frame system, moment-resisting connections between the columns and beams provide lateral resistance. In Table 12.2-1, such frames are classified as ordinary, intermediate, or special. In high Seismic Design Categories, the anticipated ground motions are expected to produce large inelastic demands so special moment frames designed and detailed for ductile response in accordance with Chapter 14 are required. In low Seismic Design Categories, the inherent overstrength in typical structural designs is such that the anticipated inelastic demands are reduced somewhat, and less ductile systems may be employed safely. Since these less ductile ordinary framing systems do not possess as much toughness, lower R values are specified.

The R, Ω_0, and C_d values for the composite systems in Table 12.2-1 are similar to those for comparable systems of structural steel and reinforced concrete. Use of the tabulated values is allowed only when the design and detailing requirements in Section 14.3 are followed.

In a dual system, a three-dimensional space frame made up of columns and beams provides primary support for gravity loads. Primary lateral resistance is supplied by shear walls or braced frames, and secondary lateral resistance is provided by a moment frame complying with the requirements of Chapter 14.

Where a beam-column frame or slab-column frame lacks special detailing, it cannot act as an effective backup to a shear wall subsystem so there are no dual systems with ordinary moment frames. Instead, Table 12.2-1 permits the use of a shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls. Use of this defined system, which requires compliance with Section 12.2.5.10, offers a significant advantage over a simple combination of the two constituent ordinary reinforced concrete systems. Where those systems are simply combined, Section 12.2.3.2 would require use of design parameters for an ordinary reinforced concrete moment frame.

In a cantilevered column system, stability of mass at the top is provided by one or more columns with base fixity acting as a single-degree-of-freedom system.

Cantilever column systems are essentially a special class of moment-resisting frame except that they do not possess the redundancy and overstrength that most moment-resisting frames derive from sequential formation of yield or plastic hinges. Where a typical moment-resisting frame must form multiple plastic hinges in members in order to develop a yield mechanism, a cantilever column system develops hinges only at the base of the columns to form a mechanism. As a result, their overstrength is limited to that provided by material overstrength and by design conservatism.

It is permitted to construct cantilever column structures using any of the systems that can be used to develop moment frames including ordinary, intermediate, and special steel and concrete detailing systems as well as timber frames. The system limitations for cantilever column systems reflect the type of moment frame detailing provided but with a height limit of 35 feet.

The R factor for cantilever column systems is derived from moment-resisting frame values where R is divided by Ω_0 but is not taken as less than 1 or greater than 3. This accounts for the lack of sequential yielding in such systems. C_d is taken as equal to R, recognizing that damping is quite low in these systems and inelastic displacement of these systems will not be less than the elastic displacement.

C12.2.2 Combinations of Framing Systems in Different Directions. Different systems can be utilized along each of the two orthogonal directions as long as the respective R, Ω_0, and C_d values are used. Depending on the combination selected, it is possible that one of the two systems will limit the extent of the overall system with regard to use and height. The more restrictive of the limitation systems governs.

C12.2.3 Combinations of Framing Systems in the Same Direction.

C12.2.3.1 R, Ω_0, and C_d Values for Vertical Combinations. The intent of the provision requiring us of the more stringent seismic design parameters (R, Ω_0, and C_d) is to prevent mixed systems that could concentrate inelastic behavior in the lower stories. Exceptions to these requirements exist for conditions that do not affect the dynamic characteristics of the structure or that will not result in concentration of inelastic demand in critical areas.

For the past several decades, building codes have allowed two-stage static analysis for certain structures with a vertical combination of dynamically uncoupled systems. While this approach may be used for any structure that meets the requirements, it is most often used for the design of light-framed construction built on a rigid concrete base. The design process requires that the "flexible" upper structure and "rigid" lower structure be designed separately with the reactions from the upper portion amplified by the ratio of respective R/ρ values. This ratio, which must be taken as no less than 1, produces demands for the "rigid" lower portion that are commensurate with its inelastic capability.

C12.2.3.2 R, Ω_0, and C_d Values for Horizontal Combinations. For nearly all conditions, the least value of R of different structural systems in the same direction must be used in design. This requirement reflects the expectation that the entire system will undergo the same deformation with its behavior controlled by the least ductile system. However, where the listed conditions are met, the R value for each independent line of resistance can be used. This exceptional condition is consistent with light-frame construction that utilizes the ground for parking with residential use above.

C12.2.4 Combination of Framing Detailing Requirements. This requirement is provided so that the higher R value system has the necessary ductile detailing throughout. The intent is that details common to both systems be designed to remain functional throughout the response in order to preserve the integrity of the seismic-force-resisting system.

C12.2.5 System Specific Requirements.

C12.2.5.1 Dual System. The moment frame of a dual system must be capable of resisting at least 25 percent of the design seismic forces; this percentage is based on judgment. The purpose of the 25 percent frame is to provide a secondary lateral system with higher degrees of redundancy and ductility in order to improve the ability of the building to support the service loads (or at least the effect of gravity loads) after strong earthquake shaking. The primary system (walls or bracing) acting together with the moment frame must be capable of resisting all of the design seismic forces. The following analyses are required for dual systems:

1. The moment frame and shear walls or braced frames must resist the design seismic forces considering fully the force and deformation interaction of the walls or braced frames and the moment frames as a single system. This analysis must be made in accordance with the principles of structural mechanics considering the relative rigidities of the elements and torsion in the system. Deformations imposed upon members of the moment frame by their interaction with the shear walls or braced frames must be considered in this analysis.
2. The moment frame must be designed with sufficient strength to resist at least 25 percent of the design seismic forces including torsional effects.

C12.2.5.2 Cantilever Column Systems. Cantilever column systems are singled out for special consideration because of their unique characteristics. These structures often have limited redundancy and overstrength and concentrate inelastic behavior at their bases. As a result, they have substantially less energy dissipation capacity than other systems. A number of apartment buildings incorporating this system experienced very severe damage and, in some cases, collapse in the 1994 Northridge earthquake. Because the ductility of columns having large axial stress is limited, cantilever column systems may not be used where column axial demands exceed 15 percent of their axial strength.

Elements providing restraint at the base of cantilever columns must be designed with overstrength so that the strength of the cantilever columns is developed.

C12.2.5.3 Inverted Pendulum-Type Structures. Inverted pendulum-type structures do not have unique entry in Table 12.2-1 since they can be formed from many structural systems. Inverted pendulum-type structures have more than half of their mass concentrated near the top (producing one degree of freedom in horizontal translation) and rotational compatibility of the mass with the column (producing vertical accelerations acting in opposite directions). Dynamic response amplifies this rotation; hence, the bending moment induced at the top of the column can exceed that computed using the procedures of Section 12.8. The requirement to design for a top moment that is one-half of the base moment calculated in accordance with Section 12.8 is based on analyses of inverted pendulums covering a wide range of practical conditions.

C12.2.5.4 Increased Building Height Limit for Steel Braced Frames and Special Reinforced Concrete Shear Walls. The first criterion for an increased building height limit precludes extreme torsional irregularity since premature failure of one of the single walls or frames could lead to excessive inelastic torsional response. The second criterion, which is similar to the redundancy requirements, is to limit the height of systems that are too strongly dependent on any single line of walls or braced frames. The inherent torsion resulting from the distance between the center or mass and center of stiffness must be included, but accidental torsional effects are neglected for ease of implementation.

C12.2.5.5 Special Moment Frames in Structures Assigned to Seismic Design Categories D through F. Special moment frames, either alone or as part of a dual system, are required to be used in Seismic Design Categories D through F where the building height exceeds 160 feet (or 240 feet for buildings that meet the provisions of Section 12.2.5.4) as indicated in Table 12.2-1. In shorter buildings where special moment frames are not required to be used, the special moment frames may be discontinued and supported on less ductile systems as long as the requirements for system combinations are followed.

For the situation where special moment frames are required, they should be continuous to the foundation. In cases where the foundation is located below the building's base, provisions for discontinuing the moment frames can be made as long as the seismic forces are properly accounted for and transferred to the supporting structure.

C12.2.5.6 Single-Story Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category D or E. Ordinary and intermediate moment frames are less ductile than special moment frames; consequently, restrictions are placed on their use in higher Seismic Design Categories. The height limit of 65 feet and the limitations on roof and wall dead load are intended to restrict the use of such systems to metal buildings and similar one-story structures, the design of which is often controlled by wind forces, and which have generally evidenced acceptable performance in past seismic events.

C12.2.5.7 Other Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category D or E. Compared to the limits in Section 12.2.5.6, this section imposes a stricter height limit because higher loads and additional stories are permitted. Low-rise light-frame structures that are commonly used in residential construction generally have evidenced adequate performance in past seismic events due to their light weight, abundance of lateral force-resisting elements, and general resilience.

C12.2.5.8 Single-Story Steel Ordinary and Intermediate Moment Frames in Structures Assigned to Seismic Design Category F. See Section C12.2.5.6.

C12.2.5.9 Other Steel Intermediate Moment Frame Limitations in Structures Assigned to Seismic Design Category F. The intent of this section is to prohibit the use of steel ordinary moment frames in light-frame construction that does not comply with Section 12.2.5.8.

C12.2.5.10 Shear Wall-Frame Interactive Systems. For structures assigned to Seismic Design Category A or B (where seismic hazard is low), it is usual practice to design shear walls and frames of a shear wall-frame structure to resist lateral forces in proportion to their relative rigidities, considering interaction between the two subsystems at all levels. As discussed in Section C12.2.1, this typical approach would require use of a lower *R* factor than that defined for shear wall-frame interactive systems. Where the special requirements of this section are satisfied, more reliable performance is expected, justifying a higher *R* factor.

C12.3 DIAPHRAGM FLEXIBILITY, CONFIGURATION IRREGULARITIES, AND REDUNDANCY

C12.3.1 Diaphragm Flexibility. Most seismic-force-resisting systems have two distinct parts: the horizontal system that distributes lateral forces to the vertical elements and the vertical system that transmits lateral forces between the floor levels and the base of the structure.

The horizontal system may consist of diaphragms or a horizontal bracing system. For the majority of buildings, diaphragms offer the most economical and positive method of resisting and distributing seismic forces in the horizontal plane. Typically, diaphragms consist of metal deck (with or without concrete), concrete slabs, and wood sheathing/decking. While most diaphragms are flat, consisting of the floors of buildings, they also may be inclined, curved, warped, or folded configurations, and most diaphragms have openings.

The diaphragm stiffness relative to the stiffness of the supporting vertical seismic-force-resisting system ranges from flexible to rigid and is important to define. Provisions defining diaphragm flexibility are given in Sections 12.3.1.1 through 12.3.1.3. If a diaphragm cannot be idealized as either flexible or rigid, explicit consideration of its stiffness must be included in the analysis.

The diaphragms in most buildings braced by wood light-frame shear walls are semi-rigid. Because semi-rigid diaphragm modeling is beyond the capability of available software for wood light-frame buildings, it is anticipated that this requirement will be met by evaluating force distribution using both rigid and flexible diaphragm models and taking the worst case of the two. While this is in conflict with common design practice, which typically includes only flexible diaphragm force distribution for wood light-frame buildings, it is one method of capturing the effect of the diaphragm stiffness.

Further detailed discussion of diaphragms can be found in Delebi, et al. (1980) and in an Applied Technology Council report on diaphragms (1981).

C12.3.1.2 Rigid Diaphragm Condition. Span length is included in the deemed-to-comply condition as an indirect measure of the flexural contribution to diaphragm stiffness.

C12.3.2 Irregular and Regular Classification. The configuration of a structure can significantly affect its performance during a strong earthquake producing the ground motion contemplated in the standard. Configuration can be divided into two aspects: horizontal and vertical. Most seismic design provisions were derived for buildings having regular configurations, but earthquakes have shown repeatedly that buildings having irregular configurations suffer greater damage. This situation prevails even with good design and construction. There are several reasons for this poor behavior of irregular structures. In a regular structure, the inelastic response produced by strong ground shaking, including energy dissipation and damage, tends to be well distributed throughout the structure. However, in irregular structures, inelastic behavior can be concentrated by irregularities and result in rapid failure of structural elements in these areas. In addition, some irregularities introduce unanticipated demands into the structure, which designers frequently overlook when detailing the structural system. Finally, the elastic analysis methods typically employed in the design of structures often cannot predict the distribution of earthquake demands in an irregular structure very well, leading to inadequate design in the areas associated with the irregularity. For these reasons, the standard encourages regular configurations and prohibits gross irregularity in buildings located on sites close to major active faults where very strong ground motion and extreme inelastic demands are anticipated.

C12.3.2.1 Horizontal Irregularity. A building may have a symmetric geometric shape without re-entrant corners or wings but still be classified as irregular in plan because of its distribution of mass or vertical seismic-force-resisting elements. Torsional effects in earthquakes can occur even where the centers of mass and resistance coincide. For example, ground motion waves acting on a skew with respect to the building axis can cause torsion. Cracking or yielding in an asymmetric fashion also can cause torsion. These effects also can magnify the torsion due to eccentricity between the centers of mass and resistance. Torsional irregularities are defined to address this concern.

A square or rectangular building with minor re-entrant corners would still be considered regular, but large re-entrant corners creating a crucifix form would produce an irregular configuration. The response of the wings of this type of building generally differs from the response of the building as a whole, and this produces higher local forces than would be determined by application of the standard without modification. Other winged plan configurations (e.g., H-shapes) are classified as irregular even if symmetric due to the response of the wings.

Significant differences in stiffness between portions of a diaphragm at a level are classified as irregularities since they may cause a change in the distribution of seismic forces to the vertical components and create torsional forces not accounted for in the distribution normally considered for a regular building. Figure C12.3-1 illustrates plan irregularities.

Where there are discontinuities in the path of lateral force resistance, the structure cannot be considered to be regular. The most critical discontinuity defined is the out-of-plane offset of vertical elements of the seismic-force-resisting system. Such offsets impose vertical and lateral load effects on horizontal elements that are difficult to provide for adequately.

Where vertical elements of the lateral-force-resisting system are not parallel to or symmetric about major orthogonal axes, the equivalent lateral force procedure of the standard cannot be applied appropriately so the structure is considered to be irregular.

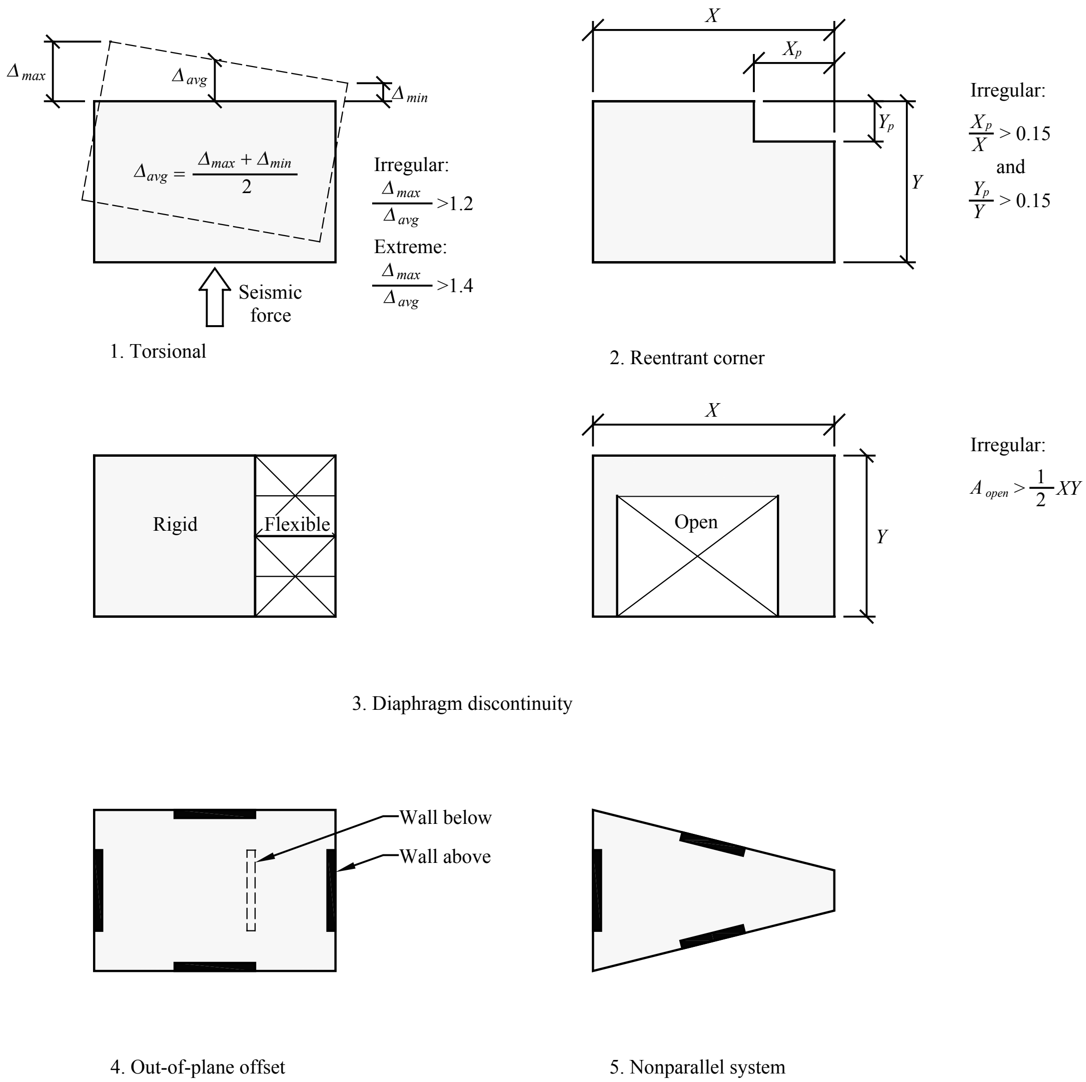


Figure C12.3-1 Building plan irregularities.

C12.3.2.2 Vertical Irregularity. Vertical configuration irregularities affect the responses at the various levels and induce loads at these levels that differ significantly from the distribution assumed in the equivalent lateral force procedure given in Section 12.8. A moment-resisting frame building might be classified as having a vertical irregularity if one story is much taller than the adjoining stories and the design did not compensate for the resulting decrease in stiffness that normally would occur. Figure C12.3-2 illustrates vertical irregularities.

A building is classified as irregular where the ratio of mass to stiffness in adjacent stories differs significantly. This might occur where a heavy mass (e.g., an interstitial mechanical floor) is placed at one level. Irregularity Type 3 in Table 12.3-2 applies regardless of whether the larger dimension is above or below the smaller one. Buildings with a weak-story irregularity tend to develop all of their inelastic behavior and consequent damage at the weak story, possibly leading to collapse. Section 12.3.3.2 provides an exception for Seismic Design Category B or C structures where essentially elastic response of the weak story is expected.

C12.3.3 Limitations and Additional Requirements for Systems with Structural Irregularities.

C12.3.3.1 Prohibited Horizontal and Vertical Irregularities in Seismic Design Categories D through F. The irregularity prohibitions of this section stem from poor performance in past earthquakes and the potential to concentrate large inelastic demands in certain portions of the structure. Even when such irregularities are permitted, they should be avoided whenever possible in all structures.

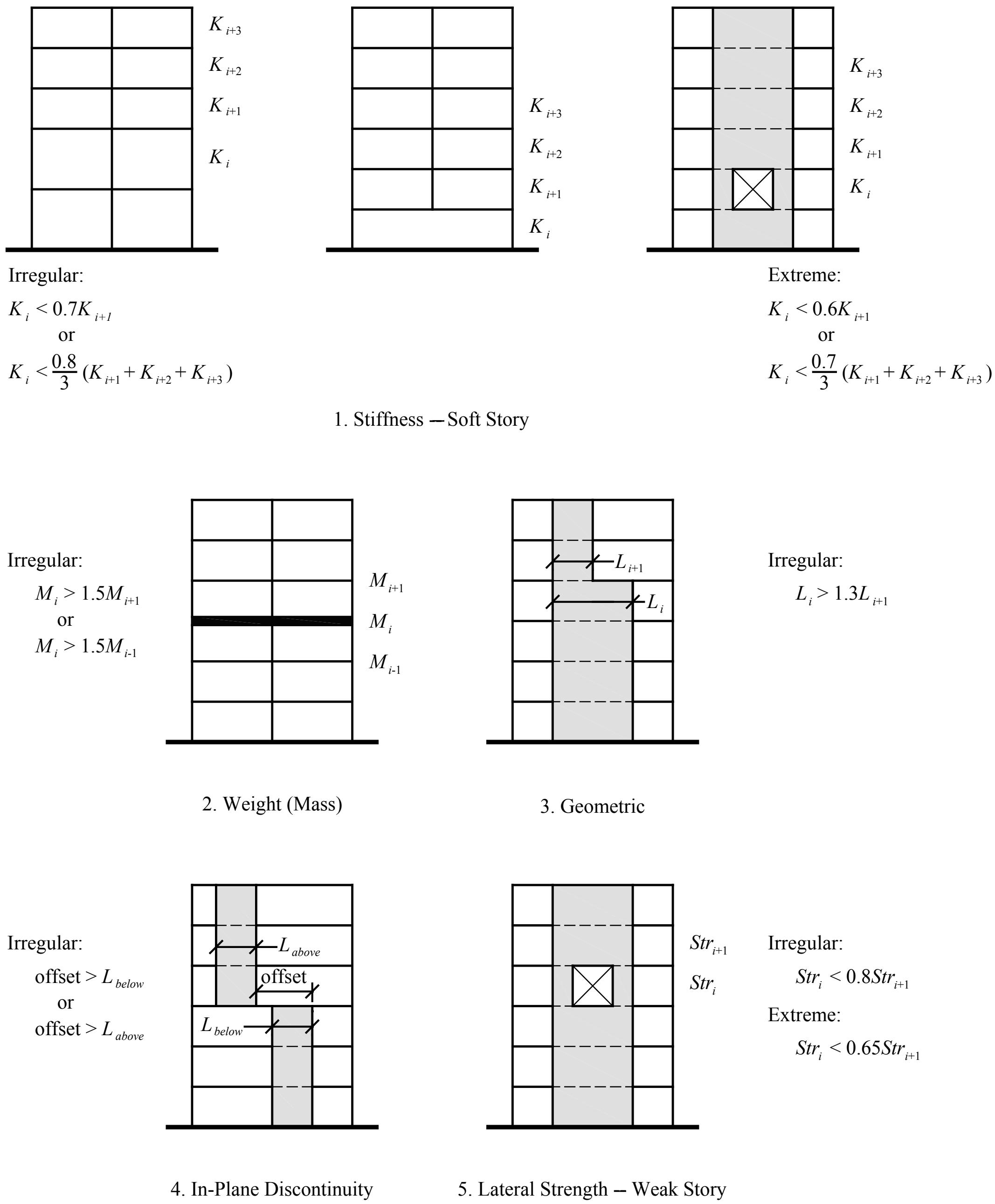


Figure C12.3-2 Building vertical irregularities.

C12.3.3.2 Extreme Weak Stories. Since extreme weak story irregularities are prohibited for buildings located in Seismic Design Categories D, E and F, the limitations and exceptions in this section apply only to buildings assigned to Seismic Design Category B or C.

C12.3.3.3 Elements Supporting Discontinuous Walls or Frames. The purpose of this requirement is to protect the supporting elements from overload caused by overstrength of a discontinued seismic-force-resisting element. Columns, beams, slabs, or trusses may be subject to such failure so all are included in the design requirement. Overload may result from forces in either the downward or upward direction; therefore, both possibilities must be considered. Such load reversals may be especially problematic for reinforced concrete beams, weaker top laminations of glulam beams, unbraced flanges of steel beams, and steel trusses.

The connection between the discontinuous element and the supporting member must be adequate to transmit the forces for which the discontinuous element is designed. For example, where the discontinuous element must be designed using the load combinations of Section 12.4.3, as is the case for a steel column in a braced frame or moment frame, its connection to the supporting member must be designed using the same load combinations. Since concrete shear walls are not required to be designed using the load combinations of Section 12.4.3, the connection between a discontinuous shear wall and the supporting member may be designed using the loads associated with the shear wall and not the load combinations with overstrength factor.

C12.3.3.4 Increase in Forces Due to Irregularities for Seismic Design Categories D through F. The irregularities listed may result in loads that are distributed differently than assumed in the equivalent lateral force procedure of Section 12.8, especially as related to the interconnection of the diaphragm with vertical elements of the seismic-force-resisting system. The 25 percent increase in force is intended to account for this difference. Where the load combinations with overstrength apply, no further increase is warranted.

C12.3.4 Redundancy. The desirability of redundancy, or multiple lateral-force-resisting load paths, has long been recognized. The redundancy provisions of this section reflect the belief that an excessive loss of story shear strength or development of an extreme torsional irregularity may lead to structural failure. The redundancy factor determined for each direction may differ.

C12.3.4.1 Conditions Where Value of ρ is 1.0. This section provides a convenient list of conditions where ρ is 1.0.

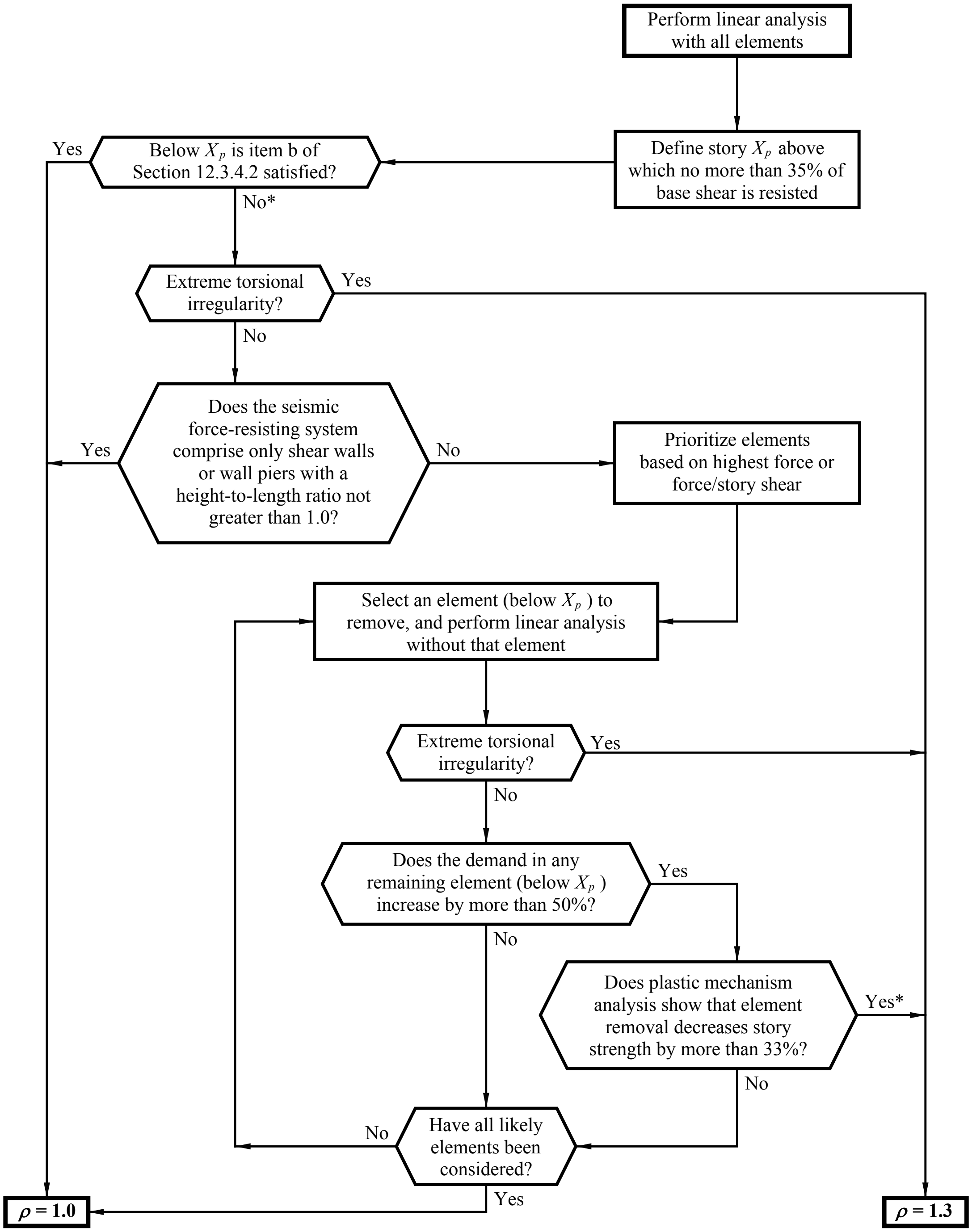
C12.3.4.2 Redundancy Factor, ρ, for Seismic Design Category D through F. There are two approaches to establishing a redundancy factor of 1.0. Where neither condition is satisfied, ρ is taken equal to 1.3. It is permitted to take ρ equal to 1.3 without checking either condition.

The first approach is a check of the elements outlined in Table 12.3-3 for cases where the story shear exceeds 35 percent of the base shear. Parametric studies (conducted by Building Seismic Safety Council Technical Subcommittee 2 but unpublished) were used to select the 35 percent value. Those studies indicated that stories with at least 35 percent of the base shear include all stories of low-rise buildings (buildings up to 5 to 6 stories) and about 87 percent of the stories of tall buildings. The intent of this limit is to exclude penthouses and the uppermost stories from the redundancy requirements.

This approach requires the removal (or loss of moment resistance) of an individual lateral-force-resisting element to determine its effect on the remaining structure. If the removal of elements, one-by-one, does not result in more than a 33 percent reduction in story strength or an extreme torsional irregularity, ρ may be taken as 1.0. For this evaluation, the determination of story strength requires an in-depth calculation. The intent of the check is to use a simple measure (elastic or plastic) to determine whether an individual member has a significant effect on the overall system. If the original structure has an extreme torsional irregularity to begin with, the resulting ρ is 1.3. Figure C12.3-3 presents a flowchart for implementing the redundancy requirements.

As indicated in the table, braced frame, moment frame, shear wall, and cantilever column systems must conform to redundancy requirements. Dual systems also are included but, in most cases, are inherently redundant. Shear walls or wall piers with a height-to-length aspect ratio greater than 1.0 within any story have been included; however, the required design of collector elements and their connections for Ω_0 times the design force may address the key issues. In order to satisfy the collector force requirements, a reasonable number of shear walls usually is required. Regardless, shear wall systems are addressed in this section so that either an adequate number of wall elements is included or the proper redundancy factor is applied. For wall piers, the height is taken as the height of the adjacent opening and generally is less than the story height.

The second approach is a deemed-to-comply condition wherein the structure is regular and has a specified arrangement of seismic-force-resisting elements to qualify for ρ of 1.0. As part of the parametric study, simplified braced frame and moment frame systems were investigated to determine their sensitivity to the analytical redundancy criteria. This simple deemed-to-comply condition is consistent with the results of the study.



* or not considered

Figure C12.3-3 Calculation of the redundancy factor, ρ.

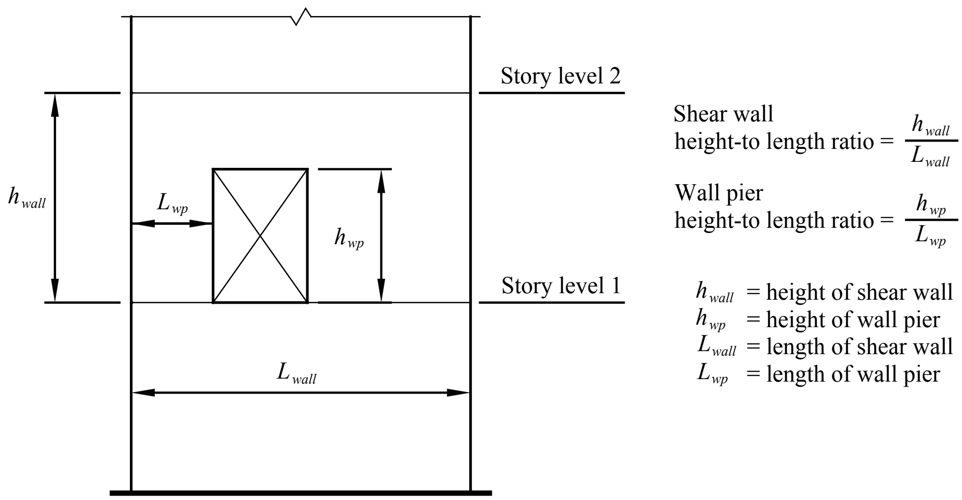


Figure C12.3-4 Shear wall and wall pier height-to-length ratios.

C12.4 SEISMIC LOAD EFFECTS AND COMBINATIONS

C12.4.1 Applicability. Structural elements designated by the engineer as part of the seismic-force-resisting system typically are designed directly for seismic load effects. None of the seismic forces associated with the design base shear are formally assigned to structural elements that are not designated as part of the seismic-force-resisting system, but such elements must be designed using the load conditions of Section 12.4 and must accommodate the deformations resulting from application of seismic loads.

C12.4.2 Seismic Load Effect. Section 12.4 presents the required combinations of seismic forces with other loads. The load combinations are taken from the basic load combinations of Chapter 2 of the standard with further elaboration of the seismic load effect, *E*. The seismic load effect includes horizontal and vertical components. For strength design, the effect of vertical seismic forces, E_v, is based on an assumed effective vertical acceleration of $0.2S_{DS}$ times gravity.

It may be helpful to recognize that the quantities E_h and E_v are the effects of loads, not the loads themselves. They can be tension or compression axial forces, shear, bending moments, or torsional moments. For a one-story shear wall, application of the horizontal seismic forces from *V* causes overturning moment and shear in the wall, both of which are E_h effects. The factor $0.2\ S_{DS}$ times gravity dead load corresponds to an E_v load effect that increases or decreases the axial force in the wall. In this simple example, an E_h force or moment is never added directly to an E_v force or moment because the former affects only moment and shear, while the latter affects only axial force.

While the shear and moment are independent of the axial force, the capacity check of the wall may need to include all three terms (or certainly moment and axial force) simultaneously.

For a diagonal brace that carries earthquake and gravity load, application of the horizontal seismic forces from *V* causes a brace force that has both horizontal and vertical components, and the factor $0.2\ S_{DS}$ times dead load produces a load effect that also affects both the horizontal and vertical components of axial force. In this case the brace force is based on $E_h \pm E_v$. Section 12.4.2.3 presents the load combinations written using the separate horizontal and vertical load effects that constitute *E*.

The $0.2S_{DS}$ vertical acceleration effect is required to be considered in the design of all members of a structure—even those that are not part of the seismic-force-resisting system. For example, design of a gravity load-resisting prestressed concrete girder may be governed by the dead and earthquake condition, where $0.2S_{DS}D$ is subtracted from the dead load. This could be the controlling condition for tension at the top of the girder.

C12.4.3 Seismic Load Effect Including Overstrength Factor. Certain structural elements or actions, such as collectors in Seismic Design Categories C through F or columns supporting discontinuous walls, are required to be designed for seismic load combinations with overstrength. In such cases the seismic load effect, E_m, has its horizontal component multiplied by the overstrength factor Ω_0, as indicated in Section 12.4.3.

C12.4.4 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Categories D through F. In Seismic Design Categories D, E, and F, horizontal cantilevers are designed for an upward force that results from an effective vertical acceleration of 1.2 times gravity. This is to provide some minimum strength in the upward direction and to account for possible dynamic amplification of vertical ground motions resulting from the vertical flexibility of the cantilever. The requirement is not applied to downward forces on cantilevers, for which the typical load combinations are used.

C12.5 DIRECTION OF LOADING

Seismic forces are delivered to a building through ground accelerations that may approach from any direction relative to the orthogonal directions of the building; therefore, seismic effects are expected to develop in both directions simultaneously. The standard requires structures to be designed for the most critical loading effects from seismic forces applied in any direction, and the procedures outlined in this section are deemed to satisfy that requirement.

The orthogonal combination procedure combines the effects from 100 percent of the seismic load applied in one direction with 30 percent of the seismic load applied in the perpendicular direction. Combining effects for seismic loads in each direction and accidental torsion results in 16 load combinations as follows:

Orthogonal load combinations	where :
$Q_E = \pm Q_{E\_X+AT} \pm 0.3Q_{E\_Y}$	Q_{E_Y} = effect of Y-direction load at the center of mass (Section 12.8.4.2)
$Q_E = \pm Q_{E\_X-AT} \pm 0.3Q_{E\_Y}$	Q_{E_X} = effect of X-direction load at the center of mass (Section 12.8.4.2)
$Q_E = \pm Q_{E\_Y+AT} \pm 0.3Q_{E\_X}$	AT = accidental torsion computed in accordance with Section 12.8.4.2
$Q_E = \pm Q_{E\_Y-AT} \pm 0.3Q_{E\_X}$	

For horizontal structural elements such as beams and slabs, orthogonal effects may be minimal; however, for vertical elements of the seismic-force-resisting system that participate in both orthogonal directions, the design likely will be governed by these combinations.

Orthogonal combinations should not be confused with modal combinations such as the square root of the sum of the squares (SRSS) or complete quadratic combination (CQC) technique.

The maximum effect of seismic forces, Q_E, from orthogonal load combinations must be modified by the redundancy factor, ρ, or the overstrength factor, Ω_0, and consider the effects of vertical seismic forces, E_V, in accordance with Section 12.4, to obtain the seismic load effect, E.

C12.6 ANALYSIS SELECTION PROCEDURE

Table 12.6-1 applies only to buildings without seismic isolation (Chapter 17) or passive energy devices (Chapter 18).

The procedures addressed in Table 12.6-1 are equivalent lateral force (ELF) analysis (Section 12.8), modal response spectrum (MRS) analysis (Section 12.9), linear response history (LRH) analysis, and nonlinear response history (NRH) analysis. Requirements for performing response history analysis are provided in Chapter 16. Nonlinear static (pushover) analysis is not addressed in the standard.

The value of T_s (= S_{D1}/S_{DS}) depends on the site class because S_{DS} and S_{D1} include such effects. Where ELF is not allowed, analysis must be performed using modal response spectrum or response history analysis.

ELF is not allowed for buildings with the listed irregularities because it assumes a gradually varying distribution of mass and stiffness along the height and negligible torsional response. The $3.5T_s$ limit recognizes that higher modes are more significant in taller buildings (Lopez and Cruz, 1996; Chopra, 2007) such that the ELF method may underestimate the design base shear and may not predict correctly the vertical distribution of seismic forces.

C12.7 MODELING CRITERIA

C12.7.1 Foundation Modeling. Structural systems consist of three interacting components: the structural framing (girders, columns, walls, diaphragms), the foundation (footings, piles, caissons), and the supporting soil. The ground motion that a structure experiences, as well as the response to that ground motion, depends on the complex interaction between these components.

Those aspects of ground motion that are affected by site characteristics are assumed to be independent of the structure-foundation system as these effects would occur in the free-field in the absence of the structure. Hence, site effects are considered separately (Sections 11.4.2 through 11.4.4 and Chapters 20 and 21).

Given a site-specific ground motion or response spectrum, the dynamic response of the structure will depend on the foundation system and on the characteristics of the soil that support the system. The dependence of the response on the structure-foundation-soil system is referred to as soil-structure interaction. Such interactions will usually, but not always, result in a reduction of base shear. This reduction in shear is due to the flexibility of the foundation-soil system and an associated lengthening of the period of vibration of the structure. In addition, the soil system may provide an additional source of damping. However, that total displacement typically increases with soil-structure interaction.

If the foundation is considered to be rigid, the computed base shears usually will be conservative, and it is for this reason that rigid foundation analysis is allowed. The designer may ignore soil-structure interaction or may consider it explicitly in accordance with Section 12.13.3 or implicitly in accordance with Chapter 19.

C12.7.2 Effective Seismic Weight. During an earthquake, the structure accelerates laterally, and these accelerations of the structural mass produce inertial forces. These inertial forces, accumulated over the height of the structure, produce the design base shear.

When a building vibrates during an earthquake, only that portion of the mass or weight that is physically tied to the structure needs to be considered as effective. Hence, live loads (e.g., loose furniture, loose equipment, and human occupants) need not be included. However, certain types of live loads such as storage loads may develop inertial forces, particularly where they are densely packed.

Also considered as effective weight is all permanently attached equipment (e.g., air conditioners, elevator equipment, and mechanical systems), movable partitions (a minimum of 10 psf is required), and 20 percent of significant roof snow load. The full snow load need not be considered because maximum snow load and maximum earthquake load are unlikely to occur simultaneously and loose snow does not move with the roof.

C12.7.3 Structural Modeling. The development of a mathematical model of a structure is always required because the story drifts and the design forces in the structure cannot be computed without such a model. In some cases, the mathematical model can be as simple as a free-body diagram as long that model can appropriately capture the strength and stiffness of the structure.

The most realistic analytical model is three-dimensional, includes all sources of stiffness (and flexibility) of the structure and the soil-foundation system as well as P-delta effects, and allows for nonlinear inelastic behavior in all parts of the structure-foundation-soil system. Development of such an analytical model is very time consuming, and such analysis is rarely warranted for typical building designs performed in accordance with the standard. Instead of performing a nonlinear analysis, inelastic effects are accounted for indirectly in the linear analysis methods by means of the response modification factor, R, and the deflection amplification factor, C_d.

Using modern software, it often is more difficult to decompose a structure into planar models than it is to develop a full three-dimensional model so three-dimensional models now are commonplace. Increased computational efficiency has reduced the motivation to model rigid diaphragms, allowing for easy and efficient modeling of diaphragm flexibility. Three-dimensional models are required where the structure has torsional irregularities, out-of-plane offset irregularities, or nonparallel system irregularities.

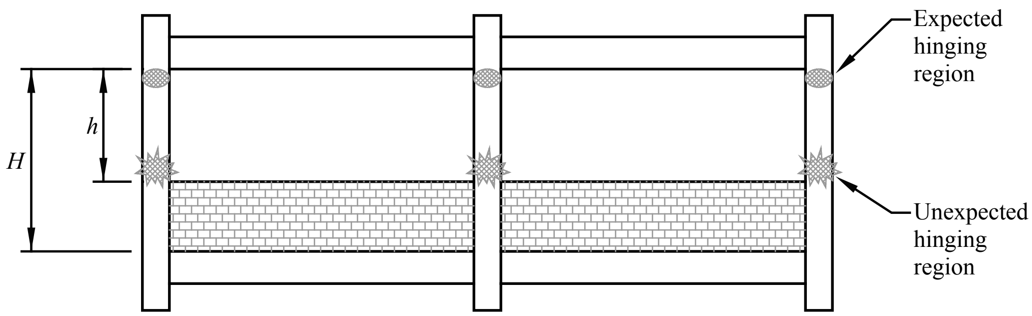
In general, the same three-dimensional model may be utilized for equivalent lateral force, modal response spectrum, and linear response history analysis. The response spectrum and linear response history models require a realistic modeling of structural mass, and the response history method also requires an explicit representation of inherent damping. Five percent critical damping is automatically included in the modal response spectrum approach. See Chapter 16 and the related commentary for additional information on linear and nonlinear response history analysis.

It is well known that deformations in the panel zones of the beam-column joints of steel moment frames are a significant source of flexibility. Two different mechanical models for including such deformations are summarized in Charney and Marshall (2006). These methods apply to both elastic and inelastic systems. For elastic structures, centerline analysis provides reasonable, but not always conservative, estimates of frame flexibility. Fully rigid end zones should not be used, as this will always result in an overestimation of lateral stiffness in steel moment-resisting frames. Partially rigid end zones may be justified in certain cases such as where doubler plates are used to reinforce the panel zone.

Including the effect of composite slabs on the stiffness of beams and girders may be warranted in some circumstances. Where composite behavior is included, due consideration should be paid to the reduction in effective composite stiffness for portions of the slab in tension (Schaffhausen and Wegmuller, 1977; Liew, et al., 2001)

For reinforced concrete buildings, it is important to address the effects of axial, flexural, and shear cracking in modeling the effective stiffness of the structural components. Determining appropriate effective stiffness of the structural components should take into consideration the anticipated demands on the components, their geometry, and the complexity of the model. Recommendations for computing cracked section properties may be found in Paulay and Priestley (1992) and similar texts.

C12.7.4 Interaction Effects. The interaction requirements are intended to prevent unexpected failures in members of moment-resisting frames. Figure C12.7-1 illustrates a typical situation where masonry infill is used, and this masonry is fitted tightly against reinforced concrete columns. Since the masonry is much stiffer than the columns, column hinges form at the top of column and at the top of the masonry rather than at the top and bottom of the column. If the column flexural capacity is M_p, the shear in the columns increases by the factor H/h, and this may cause an unexpected nonductile shear failure in the columns. Many building collapses have been attributed to this effect.



Expected plastic hinge capacity = M_p

Expected column shear = $2M_p/H$

Actual column shear = $2M_p/h$

Figure C12.7-1 Undesired interaction effects.

C12.8 EQUIVALENT LATERAL FORCE PROCEDURE

The equivalent lateral force (ELF) procedure provides a simple way to incorporate the effects of inelastic dynamic response into a linear static analysis. This procedure is useful in preliminary design of all structures and is allowed for final design of the vast majority of structures. The procedure is valid only for structures without significant discontinuities in mass and stiffness along the height, where the dominant response to ground motions is in the horizontal direction without significant torsion.

The ELF procedure has three basic steps:

1. Determine the seismic base shear,
2. Distribute the shear vertically along the height of the structure, and
3. Distribute the shear horizontally across the width and breadth of the structure.

Each of these steps is based on a number of simplifying assumptions. A broader understanding of these assumptions may be obtained from any structural dynamics textbook that emphasizes seismic applications.

C12.8.1 Seismic Base Shear

C12.8.1.1 Calculation of Seismic Response Coefficient. Equation 12.8-1 simply expresses the base shear as the product of the effective seismic weight, W, and a response coefficient, C_s. The response coefficient is a spectral pseudoacceleration, in g units, which has been modified by R and I to account for inelastic behavior and to provide for improved performance for high occupancy or essential structures.

There are five equations for determining the response coefficient C_s; the first three are plotted in Figure C12.8-1.

Equation 12.8-2, representing the constant acceleration part of the spectrum, controls where $0.0 < T < T_s$. As shown in Table C12.6-1 (which provides values of $3.5T_s$), T_s is a function of seismicity and site. It may be as low as 0.2 seconds for low hazard regions on Site Class B or as high as 0.9 seconds in high hazard regions on Site Class E.

The true pseudoacceleration response spectrum transitions to the peak ground acceleration as the period approaches zero. This transition is not used in the ELF method. One reason is that simple reduction of the response spectrum by ($1/R$) in the very short period region would exaggerate inelastic effects.

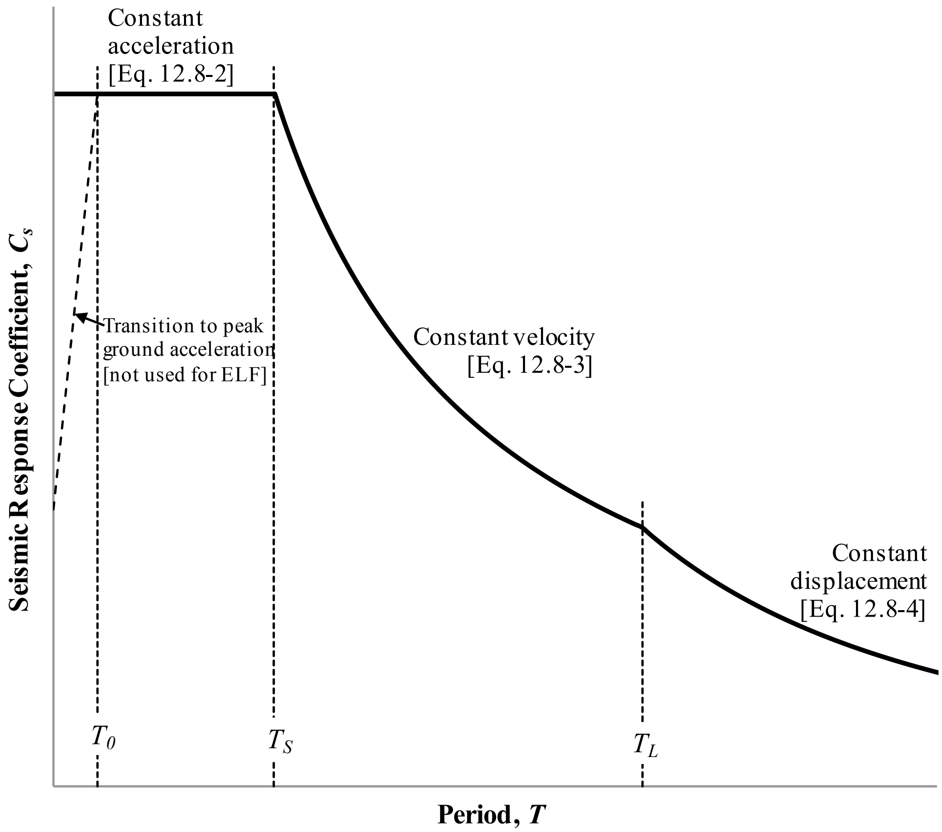


Figure C12.8-1 Seismic response coefficient versus period.

Equation 12.8-3, representing the constant velocity part of the spectrum, controls where $T_s < T < T_L$. In this region, the seismic response coefficient is inversely proportional to period, and the pseudovelocity (pseudoacceleration divided by circular frequency, ω, is constant. $T_{L,}$ the long-period transition period, is provided in Figures 22-15 through 22-20. T_L ranges from 4 seconds in in the northcentral conterminous states and western Hawaii to 16 seconds in the Pacific Northwest and in western Alaska.

Equation 12.8-4, representing the constant displacement part of the spectrum, controls where $T > T_L$. Given the current mapped values of T_L, this equation only affects tall and flexible structures.

Equation 12.8-5 is the minimum base shear and provides a (working stress) strength of approximately 3 percent of the weight of the structure (Seismology Committee, Structural Engineers Association of California, 1996). This minimum base shear was originally enacted in 1933 by the state of California's Riley Act.

Equation 12.8-6 applies to sites near major active faults (as reflected by values of S_1) where pulse effects can increase long-period demands.

C12.8.1.2 Soil-Structure Interaction Reduction. Soil-structure interaction, which can influence significantly the dynamic response of structures to earthquakes, is addressed in Chapter 19.

C12.8.1.3 Maximum S_s Value in Determination of C_s. The maximum value of S_s was created as hazard maps were revised in 1997. The cap on S_s reflects engineering judgment about performance of code-complying buildings in past earthquakes so the height, period, and regularity conditions required for use of the limit are very important qualifiers.

C12.8.2 Period Determination. The fundamental period of the structure, T, is used to determine the design base shear as well as the exponent k that establishes the distribution of the shear along the height of the structure. Equation 12.8-7 is an empirical relationship determined through statistical analysis of the measured response of buildings in California. Figure C12.8-2 illustrates such data for various structures with steel moment resisting frames.

Since the empirical expression is based on the lower bound of the data, it produces a lower bound for the period of a building of given height. This lower bound period, used in Equations 12.8-3 and 12.8-4, provides a conservative estimate of base shear.

The fundamental period determined from a rational analysis may be used in design unless it exceeds the approximate period times the coefficient provided in Table 12.8-1. This period limit prevents the use of unusually low ELF base shear for design of buildings (or computational models) that are overly flexible. The coefficients in the table have two effects. First, the conservatism of lower bound empirical formulas for T_a is removed. Second, the period is increased in regions of lower seismicity as buildings in such areas generally are more flexible (and, hence, have longer periods) than buildings in regions of higher seismicity.

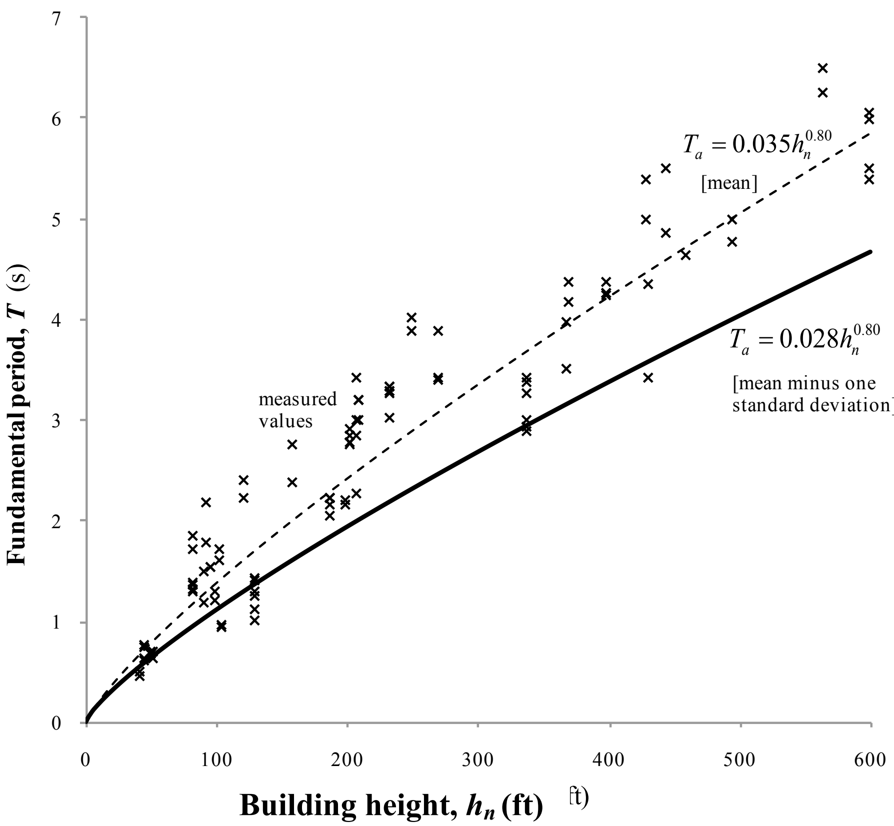
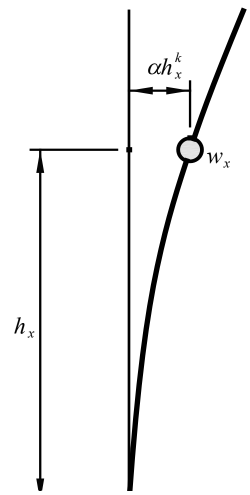


Figure C12.8-2 Variation of fundamental period with building height.

C12.8.3 Vertical Distribution of Seismic Force. Equation 12.8-12 is based on the simplified first mode shape shown in Figure C12.8-3. In the figure, F_x is the inertial force at level *x*, which is simply the total acceleration at level *x* times the mass at level *x*. The base shear is the sum of these inertial forces, and Equation 12.8 simply gives the ratio of the force at level *x* to the total base shear.

The deformed shape of the structure of Figure C12.8-3 is a function of the exponent *k*, which is related to the fundamental period of vibration of the structure. The variation of *k* with *T* is illustrated in Figure C12.8-4. The exponent *k* is intended to approximate the effect of higher modes, which are generally more dominant in structures with a longer fundamental period of vibration. Lopez and Cruz (1996) discuss the factors that influence higher modes of response. Although the actual first mode shape for a structure is also a function of the type of seismic-force-resisting system, that effect is not reflected in these equations.

The horizontal forces computed using Equation 12.8-12 do not reflect the actual inertial forces imparted on a structure at any particular time. Instead, they are intended to provide design story shears that are consistent with enveloped results from more accurate analysis (Chopra and Newmark, 1980).



$$F_x = \omega^2 \alpha h_x^k \frac{w_x}{g}$$

$$V_b = \sum_{i=1}^{n} \omega^2 \alpha h_i^k \frac{w_i}{g}$$

$$C_{vx} = \frac{F_x}{V_b} = \frac{w_x h_x^k}{\sum^{n} w_i h_i^k}$$

Figure C12.8-3 Basis of Equation 12.8-12.

C12.8.4 Horizontal Distribution of Forces. Within the context of an elastic ELF analysis, the distribution of lateral forces to various seismic-force-resisting elements depends on the type, geometric arrangement, and vertical extents of the resisting elements and on the shape and flexibility of the floor diaphragms. Because seismic-force-resisting elements are expected to respond inelastically to design ground motions, the distribution of forces to the various elements also depends on the strength of the elements and their sequence of yielding. Clearly, such effects cannot be captured accurately by a linear elastic static analysis (Paulay, 1997). Nonlinear dynamic analysis is too cumbersome to be applied to the design of most buildings so other approximate methods are used.

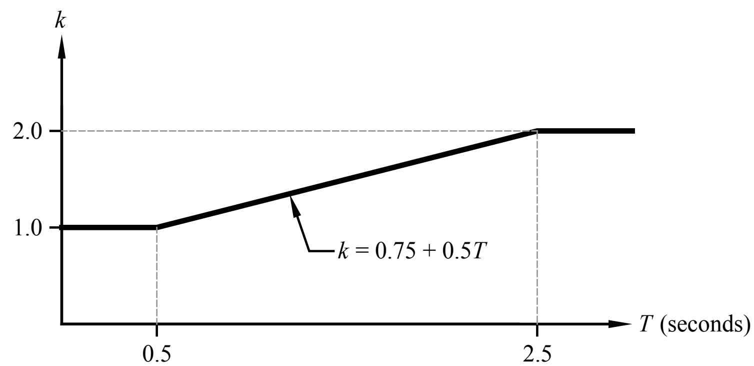


Figure C12.8-4 Variation of exponent *k* with period *T*.

Of particular concern is the torsional response of the structure during the earthquake. This response has been observed in structures that are designed to be nearly symmetric in plan and layout of seismic-force-resisting systems (De La Liera and Chopra, 1994). This torsional response is due to a variety of “accidental” eccentricities that exist due to uncertainties in quantifying the mass and stiffness distribution of the structure, as well as torsional components of ground motion that are not included explicitly in code-based designs (Newmark and Rosenbleuth, 1971).

C12.8.4.1 Inherent Torsion. When lateral forces in a particular direction are applied statically at each story of a building with rigid diaphragms, torsional displacement (twisting about the vertical axis) occurs if the centers of stiffness and mass of each story are not perfectly coincident in plan. When three-dimensional analysis is used, this inherent torsion is included automatically. When planar analysis is used, the centers of mass and rigidity for each story must be determined explicitly. Unfortunately, it is difficult to determine the center of rigidity for a multistory building to compute the inherent torsion; the center of rigidity for a particular story depends on the configuration of the seismic-force-resisting elements above and below that story and may be load dependent (Chopra and Goel, 1991).

For buildings with fully flexible diaphragms (as defined in Section 12.3), vertical elements are assumed to resist inertial forces from the mass that is tributary to the elements, but with no explicitly computed torsion. No diaphragm is perfectly flexible, so some torsional forces always develop even when they are ignored.

C12.8.4.2 Accidental Torsion. Even for perfectly symmetric buildings, the true locations of the centers of mass and rigidity are uncertain. As discussed in Section C12.8.4, other effects also may produce torsion. The requirement to consider accidental torsion is intended to address this concern.

Accidental and inherent torsions result in forces that must be combined with those obtained from the application of the lateral story forces; all components must be designed for the maximum effects determined considering positive accidental torsion, negative accidental torsion, and no accidental torsion.

C12.8.4.3 Amplification of Accidental Torsion. Equation 12.8-14 was developed by the SEAOC "seismology committee to encourage buildings with good torsional stiffness" (Structural Engineers Association of California, 1999).

In calculating the torsional amplification factor, A_x, the applied loads include inherent and accidental torsion, but with no further amplification; the calculation is not iterative.

Figure C12.8-5 illustrates the effect of Equation 12.8-14 for a symmetric rectangular building with various aspect ratios (L/B) where the seismic-force-resisting elements are positioned at a variable distance (defined by α) from the center of mass in each direction. Each element is assumed to have the same stiffness. The structure is loaded parallel to the short direction with an eccentricity of $0.05L$.

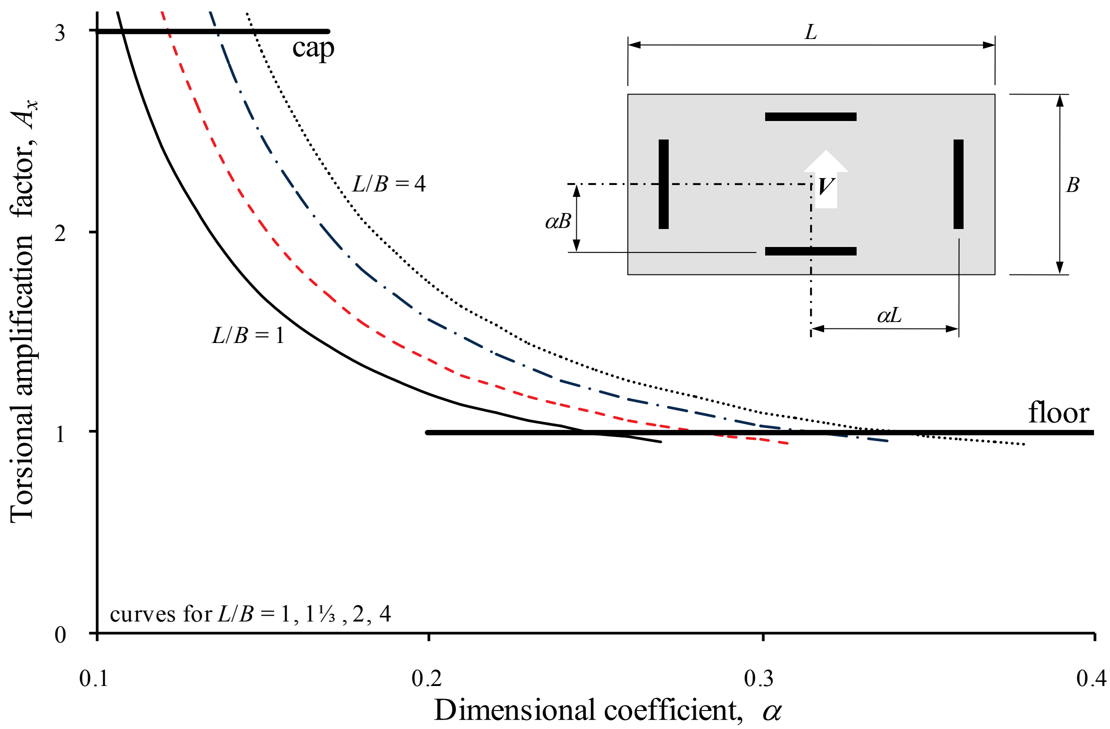


Figure C12.8-5 Amplification factor for symmetric rectangular buildings.

For α equal to 0.5, these elements are at the perimeter of the buildings, and for α equal to 0.0, they are at the center (providing no torsional resistance). For a square building (L/B = 1.00), the torsional amplification factor is greater than 1.0 where α is less than 0.25 and increases to the maximum of 3.0 where α is equal to 0.11. For a rectangular building with L/B equal to 4.00, the amplification factor is greater than 1.0 where α is less than 0.34 and increases to 3.0 where α is equal to 0.15. For the range of aspect ratios investigated, A_x is equal to 1.0 where α is greater than 0.34 and A_x reaches its maximum value of 3.0 where (α < 0.11 to 0.15).

C12.8.6 Story Drift Determination. Equation 12.8-15 is used to estimate inelastic deflections, which are then used to calculate design story drifts. These story drifts must be less than the allowable story drifts of Table 12.12-1. For buildings without torsional irregularity, computations are performed using deflections at the centers of mass of adjacent stories. For Seismic Design Category C, D, E, or F structures that are torsionally irregular, Section 12.12.1 requires that drifts be computed along the edges of the structure.

The term C_d in Equation 12.8-15 amplifies the displacements from elastic analysis at design level forces, which are reduced by R.

Figure C12.8-6 illustrates the relationships between elastic response; response to reduced design-level forces; and the expected inelastic response. If the structure remained elastic during an earthquake, the force would be V_E, and the corresponding displacement would be δ_E. Note that V_E does not include the reduction factor, R, which accounts primarily for ductility and overstrength. According to the equal displacement "rule" of seismic design, the maximum displacement of an inelastic system is approximately equal to that of an elastic system with the same initial stiffness. This condition has been observed for structures idealized with bilinear inelastic response and a fundamental period greater than T_s. For shorter period structures, peak displacement of an inelastic system tends to exceed that of the corresponding elastic system. Since the forces used for design include the response modification coefficient, R, the resulting displacements are too small and must be amplified.

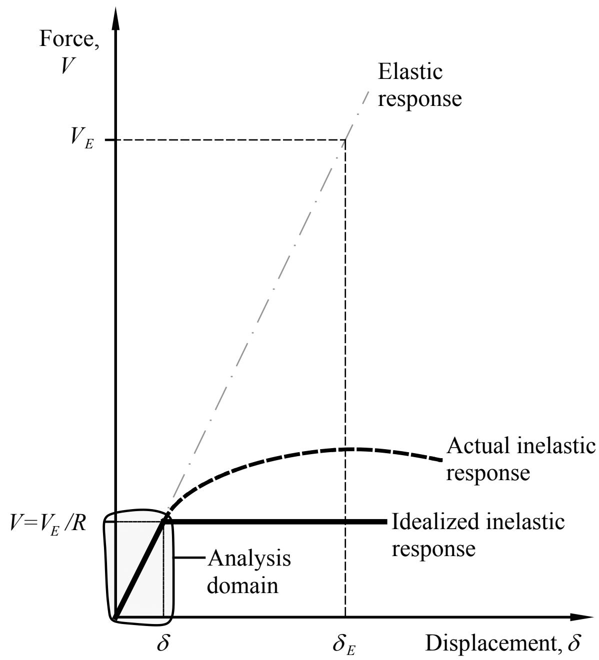


Figure C12.8-6 Displacements used to compute drift.

This analysis domain is shown in Figure C12.8-6. Because of overstrength and associated stiffness increases, the actual inelastic response differs from the idealized inelastic response; the actual displacement of the system may be less than R times δ. The standard accounts for this difference by multiplying the fictitious (design-level) elastic displacements δ by the factor C_d, which is usually less than R.

The design forces used to compute δ_{xe} include the importance factor, I, so Equation 12.8-15 includes I in the denominator. This is appropriate since the allowable story drifts (except for masonry shear wall structures) in Table 12.12-1 are more stringent for higher occupancy categories.

C12.8.6.1 Minimum Base Shear for Computing Drift. Except for period limits (as described in Section C12.8.6.2), all of the requirements of Section 12.8 (including minimum base shears and force distributions) must be satisfied where computing drift for ELF analysis.

C12.8.6.2 Period for Computing Drift. Where the response spectrum of Section 11.4.5 or the corresponding equations of Section 12.8.1 are used and the structural period is less than TL, displacements increase with increasing period (even though forces may decrease). Section 12.8.2 applies a period limit so that design forces are not too low, but if the lateral forces used to compute drifts are inconsistent with the forces corresponding to the computed period, displacements will be overestimated. Therefore, the standard allows the determination of drift using forces that are consistent with the computed period of vibration of the structure.

Computed periods greater than C_uT_a are common, particularly for moment frames. In such cases the seismic design forces used to proportion strength may produce displacements that violate drift limits, whereas displacements based on the computed period will satisfy drift limits.

The more flexible the structure, the more likely it is that P-delta effects will ultimately control the design. Computed periods that are significantly greater than (perhaps more than 1.5 times) $C_u T_a$ may indicate a modeling error.

C12.8.7 P-delta Effects. P-delta effects influence both the stiffness and strength of structures. Figure C12.8-7 shows idealized static force-displacement responses for a simple, one-story structure (such as a cantilevered column). The stiffness and strength of the structure without considering P-delta effects (condition 0) are represented by K_0 and V_0. When P-delta effects are considered (condition 1), the related quantities are K_1 and V_1. Since the two model conditions are for the same structure, inherent capacity of the structure is the same in either condition, the yield displacement is the same ($\delta_{0y} = \delta_{1y} = \delta_y$).

The geometric stiffness of the structure, K_G, is equal to P/h, where P is the total gravity load and h is the story height. K_G is negative where gravity loads cause compression in the story.

The stability coefficient, θ, is defined as the absolute value of the geometric stiffness divided by the elastic stiffness. From Figure C12.8-7, $K_0 = V_{0y} / \delta_{0y}$. Hence,

$$\theta = \frac{|K_G|}{K_0} = \left| \frac{P\delta_{0y}}{V_{0y}h} \right| \qquad \text{C12.8-1}$$

Given the above, and the geometric relationships shown in Figure C12.8-7, it can be shown that the force producing yield in condition 1 (with P-delta effects) is

$$V_{1y} = V_{0y}(1-\theta) \qquad \text{C12.8-2}$$

and that for an applied force, V, less than or equal to V_{1y}

$$\delta_1 = \frac{\delta_0}{1-\theta} \qquad \text{C12.8-3}$$

As θ approaches 1.0, δ_1 approaches infinity and V_1 approaches zero, defining a state of static instability.

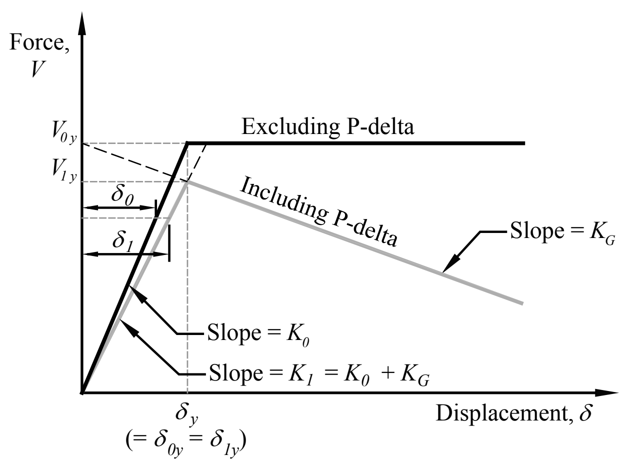


Figure C12.8-7 P-delta effect on a simple structure.

The intent of Section 12.8.7 is to determine whether P-delta effects are significant, and if so, to modify the strength and stiffness of the structure to account for such effects. Also, maximum permitted values of θ are established.

Equation 12.8-16 is used to determine the stability coefficient of each story of a structure. Where the stability coefficient exceeds 0.1, P-delta effects must be considered using one of two approaches. Displacements and member forces are either multiplied by $1/(1-\theta)$ to reflect the conditions shown in Figure C12.8-7 in accordance with the equal displacement rule or determined by rational analysis. Two types of rational analysis are envisioned. First, a nonlinear static (pushover) analysis could be performed to show that the post-yield slope of the pushover curve is continuously positive up to the target displacement. Second, a nonlinear dynamic response history analysis could be repeated with and without P-delta effects to determine if the behavior including P-delta meets all performance criteria.

Although the P-delta procedures in the standard reflect the simple static idealization shown in Figure C12.8-7, the real issue is one of dynamic stability. For that reason, nonlinear response history analysis is appealing. Such analysis should reflect variability of ground motions and system properties, including initial stiffness, strain hardening stiffness, initial strength, hysteretic behavior, and magnitude of gravity load. Unfortunately, the dynamic response of structures is highly sensitive to such parameters, causing considerable dispersion to appear in the results (Vamvatsikos, 2002). This dispersion, which increases dramatically with stability coefficient θ, is due primarily to the incrementally increasing residual deformations (ratcheting) that occur during the response. Residual deformations may be controlled by increasing either the initial strength or the secondary stiffness. See Gupta and Krawinkler (2000) for additional information.

Equation 12.8-17 establishes the maximum stability coefficient permitted. The intent of this requirement is to protect structures from the possibility of stability failures triggered by post-earthquake residual deformation.

C12.9 MODAL RESPONSE SPECTRUM ANALYSIS

In the modal response spectrum analysis method, the structure is decomposed into a number of single-degree-of-freedom systems, each having its own mode shape and natural period of vibration. The number of modes available is equal to the number of mass degrees of freedom of the structure, so the number of modes can be reduced by eliminating mass degrees of freedom. For example, rigid diaphragm constraints may be used to reduce the number of mass degrees of freedom to one per story for planar models, and to three per story (two translations and rotation about the vertical axis) for three-dimensional structures. However, where the vertical elements of the seismic-force-resisting system have significant differences in lateral stiffness, rigid diaphragm models should be used with caution as relatively small in-plane diaphragm deformations can have a significant effect on the distribution of forces.

For a given direction of loading, the displacement in each mode is determined from the corresponding spectral acceleration, modal participation, and mode shape. Because the sign (positive or negative) and the time of occurrence of the maximum acceleration are lost in creating a response spectrum, there is no way to recombine modal responses exactly. However, statistical combination of modal responses produces reasonably accurate estimates of displacements and component forces. The loss of signs for computed quantities leads to problems in interpreting force results where seismic effects are combined with gravity effects, produces forces that are not in equilibrium, and makes it impossible to plot deflected shapes of the structure.

C12.9.1 Number of Modes. The key motivation to perform modal response spectrum analysis is to determine how the actual distribution of mass and stiffness of a structure affects the elastic displacements and component forces. Where at least 90 percent of the model mass participates in the response, the distribution of forces and displacements is sufficient for design. The scaling required by Section 12.9.4 controls the overall magnitude of design values so that incomplete mass participation does not produce unconservative results.

The number of modes required to achieve 90 percent modal mass participation is usually a small fraction of the total number of modes. See Lopez and Cruz (1996) for further discussion of the number of modes to use for modal response spectrum analysis.

C12.9.2 Modal Response Parameters. The design response spectrum (whether the general spectrum from Section 11.4.5 or a site-specific spectrum determined in accordance with Section 21.2) is representative of linear elastic structures. Division of the spectral ordinates by R accounts for inelastic behavior, and multiplication of spectral ordinates by I provides the additional strength needed to improve the performance of important structures. The displacements that are computed using the response spectrum that has been modified by R and I (for strength) must be amplified by C_d and reduced by I to produce the expected inelastic displacements. (See Section C12.8.6.)

C12.9.3 Combined Response Parameters. Most computer programs provide for either the SRSS or the CQC method (Wilson, et al., 1981) of modal combination. The two methods are identical where applied to planar structures, or where zero damping is specified for the computation of the cross-modal coefficients in the CQC method. The modal damping specified in each mode for the CQC method should be equal to the damping level that was used in the development of the response spectrum. For the spectrum in Section 11.4.5, the damping ratio is 0.05.

The SRSS or CQC method is applied to loading in one direction at a time. Where Section 12.5 requires explicit consideration of orthogonal loading effects, the results from one direction of loading may be added to 30 percent of the results from loading in an orthogonal direction. Wilson (2000) suggests that a more accurate approach is to use the SRSS method to combine 100 percent of the results from each of two orthogonal directions where the individual directional results have been combined by SRSS or CQC, as appropriate.

C12.9.4 Scaling Design Values of Combined Response. The modal base shear, V_t, may be less than the ELF base shear, V, because: (a) the calculated fundamental period may be longer than that used in computing V, (b) the response is not characterized by a single mode, and (c) the ELF base shear assumes 100 percent mass participation in the first mode, which is always an overestimate. The scaling required by Section 12.9.4 provides, in effect, a minimum base shear for design. This minimum base shear is provided because the computed period of vibration may be the result of an overly flexible (incorrect) analytical model. The possible 15 percent reduction in design base shear may be considered as an incentive for using a modal response spectrum analysis in lieu of the equivalent lateral force procedure.

Displacements from the modal response spectrum are not scaled because the use of an overly flexible model will result in conservative estimates of displacement that need not be further scaled.

C12.9.5 Horizontal Shear Distribution. Accidental torsion must be included in the analysis as specified in Section 12.8.7. For modal analysis there are two basic approaches to include accidental torsion.

The first approach is to perform static analyses with accidental torsions applied at each level of the structure, and then add these results to those obtained from the modal response spectrum analysis. Where this approach is used, torsional amplification in accordance with Section 12.8.4.3 is required.

The second approach, which applies only to three-dimensional analysis, is to offset the centers of mass of each story 5 percent in each direction, thus requiring four separate models. The advantage of this method is that the effects of direct loading and accidental torsion are combined automatically. A practical disadvantage is the increased bookkeeping for multiple analyses.

Where this approach is used, further amplification of accidental torsion is not required because repositioning the center of mass in a dynamic analysis changes the natural mode shapes and frequencies, producing torsions larger than the static accidental torsion.

C12.9.6 P-delta Effects. The requirements of Section 12.8.7, including the stability coefficient limit, θ_{max}, apply to modal response spectrum analysis.

Amplification of displacements and member forces as a result of P-delta effects may be accomplished through use of the geometric stiffness. For the purpose of dynamic analysis, the linearized geometric stiffness, which includes the story-wise P-Δ effect, is usually sufficient. Using the consistent geometric stiffness (P-δ effect), which is associated with the deflected shape of the individual elements of the structure, slightly improves accuracy. Including P-delta effects directly in dynamic analysis lengthens of the periods of vibration of each mode of response and increases lateral displacements.

C12.10 DIAPHRAGMS, CHORDS, AND COLLECTORS

C12.10.1 Diaphragm Design. Diaphragms are generally treated as horizontal deep beams or trusses that distribute lateral forces to the vertical elements of the seismic-force-resisting system. As deep beams, diaphragms must be designed to resist the resultant shear and bending stresses. Diaphragms are commonly compared to girders, with the roof or floor deck analogous to the girder web in resisting shear, and the boundary elements (chords) analogous to the flanges of the girder in resisting flexural tension and compression. As in girder design, the chord members (flanges) must be sufficiently connected to the body of the diaphragm (web) to prevent separation and to force the diaphragm to work as single unit.

Diaphragms may be considered flexible, semi-rigid, or rigid. The flexibility or rigidity of the diaphragm determines how lateral forces will be distributed to the vertical elements of the seismic-force-resisting system. See Section C12.3.1. Once the distribution of lateral forces is determined, shear and moment diagrams are used to compute the diaphragm shear and chord forces. Where diaphragms are not flexible, inherent and accidental torsion must be considered in accordance with Section 12.8.4.

Diaphragm openings may require additional localized reinforcement (sub-chords and collectors) to resist the subdiaphragm chord forces above and below the opening and to collect shear forces where the diaphragm depth is reduced. (See Figure C12.10-1.) Collectors on each side of the opening drag shear into the subdiaphragms above and below the opening. The subchord and collector reinforcement must extend far enough into the adjacent diaphragm to develop the axial force through shear transfer. The required development length is determined by dividing the axial force in the sub-chord by the shear capacity (in force/unit length) of the main diaphragm.

Chord reinforcement at reentrant corners must extend far enough into the main diaphragm to develop the chord force through shear transfer. (See Figure C12.10-2.) Continuity of the chord members also must be considered where the depth of the diaphragm is not constant.

In wood and metal deck diaphragm design, framing members are often used as continuity elements, serving as sub-chords and collector elements at discontinuities. These continuity members also are often used to transfer wall out-of-plane forces to the main diaphragm, where the diaphragm itself does not have the capacity to resist the anchorage force directly. For additional discussion, see Section C12.11.2.2.3.

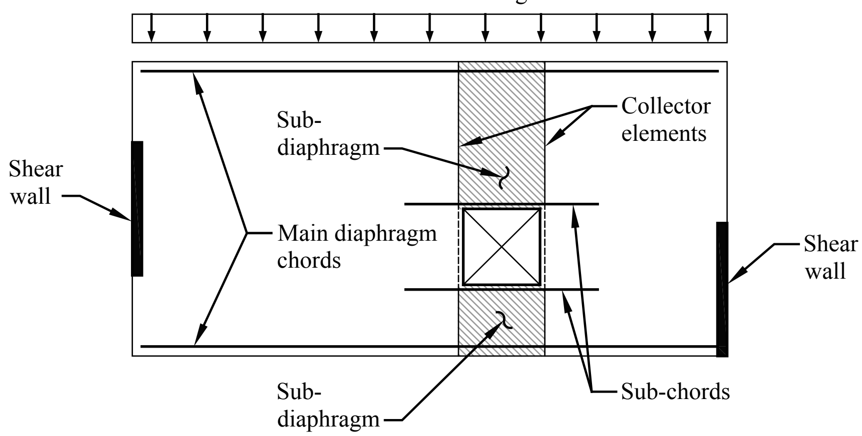


Figure C12.10-1 Diaphragm components.

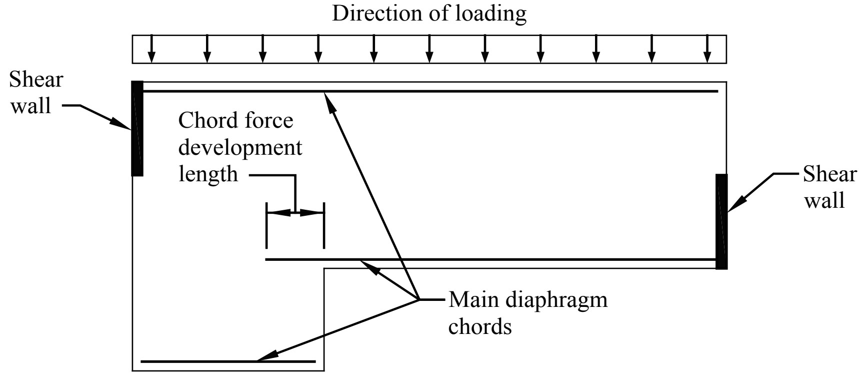


Figure C12.10-2 Diaphragm with a re-entrant corner.

C12.10.1.1 Diaphragm Design Forces. Diaphragms must be designed to resist inertial forces, as specified in Equation 12.10-1, and to transfer design seismic forces due to horizontal offsets or changes in stiffness of the vertical resisting elements. Inertial forces are those seismic forces that originate at the specified diaphragm level, while the transfer forces originate above the specified diaphragm level. The redundancy factor, ρ, used for design of the seismic-force-resisting elements also applies to diaphragm transfer forces, thus completing the load path.

C12.10.2.1 Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through F. The overstrength requirement of this section is intended to keep inelastic behavior in the ductile elements of the seismic-force-resisting system (consistent with the R factor) rather than in collector elements.

C12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

As discussed in Section C11.7, structural integrity is important not only in earthquake-resistant design but also in resisting high winds, floods, explosion, progressive failure, and even such ordinary hazards as foundation settlement. The detailed requirements of this section address wall-to-diaphragm integrity.

C12.11.1 Design for Out-of-Plane Forces. Because they are often subjected to local deformations caused by material shrinkage, temperature changes, and foundation movements, wall connections require some degree of ductility in order to accommodate slight movements while providing the required strength.

Although nonstructural walls are not subject to this requirement, they must be designed in accordance with Chapter 13.

C12.11.2 Anchorage of Concrete or Masonry Structural Walls. One major hazard in past earthquakes is the separation of heavy masonry or concrete walls from floors or roofs. The forces defined in this section apply only to the anchorage or connection of the wall to the structure, and not to overall wall design. The anchorage force should be considered both for tension (out-of-plane) and sliding (in-plane) directions.

Where the lateral spacing of connections used to resist the wall anchorage force are spaced further apart than 4 feet (1219 mm) as measured along the length of the wall, the section of wall that spans between the anchors must be designed to resist the local out-of-plane bending caused by this force.

C12.11.2.1 Anchorage of Concrete or Masonry Structural Walls to Flexible Diaphragms. Diaphragm flexibility can amplify out-of-plane accelerations so the wall anchorage forces in this condition are twice those defined in Section 12.11.1.

C12.11.2.2 Additional Requirements for Diaphragms in Structures Assigned to Seismic Design Categories C through F.

C12.11.2.2.1 Transfer of Anchorage Forces into Diaphragm. This requirement, which aims to prevent the diaphragm from tearing apart during strong shaking by requiring transfer of anchorage forces across the complete depth of the diaphragm, was prompted by failures of connections between tilt up concrete walls and wood panelized roof systems in the 1971 San Fernando earthquake. An exception is provided for modestly proportioned diaphragms of light-frame construction, which have not performed poorly.

Depending upon diaphragm shape and member spacing, numerous suitable combinations of subdiaphragms and continuous tie elements and smaller sub-subdiaphragms connecting to larger subdiaphragm and continuous tie elements are possible. The configuration of each subdiaphragm (or sub-subdiaphragm) provided must comply with the simple 2.5-to-1 length-to-width ratio, and the continuous ties must have adequate member and connection strength to carry the accumulated wall anchorage forces.

C12.11.2.2.2 Steel Elements of Structural Wall Anchorage System. A multiplier of 1.4 has been specified for strength design of steel elements in order to obtain a fracture strength of almost 2 times the specified design force (where φ_t is 0.75 for tensile rupture).

C12.11.2.2.3 Wood Diaphragms. Material standards for wood structural panel diaphragms permit the sheathing to resist shear forces only; use to resist direct tension or compression forces is not permitted. Therefore, seismic anchorage forces from walls must be transferred into framing members (such as beams, purlins, or subpurlins) using suitable straps or anchors. For wood diaphragms, it is common to use local framing and sheathing elements as subdiaphragms to transfer the uniform lateral wall forces into more concentrated lines of drag or continuity framing that carry the forces across the diaphragm and hold the building together. Figure C12.11-1 shows a schematic plan of typical roof framing using subdiaphragms.

Fasteners to wood framing are intended to transfer shear forces only along the wood framing; any forces acting transverse to the framing tend to induce splitting (due to cross-grain tension). Fasteners into wood ledgers attached to concrete or masonry walls are designed to resist shear forces only; separate straps or anchors generally are provided to transfer out-of-plane wall forces into perpendicular framing members.

C12.11.2.2.4 Metal Deck Diaphragms. In addition to transferring shear forces, metal deck diaphragms often can resist direct axial forces in at least one direction. However, corrugated metal decks cannot transfer axial forces in the direction perpendicular to the corrugations and are prone to buckling if the unbraced length of the deck as a compression element is large. To manage diaphragm forces perpendicular to the deck corrugations, it is common that metal decks are supported at 8- to 10-foot intervals by joists that are connected to walls in a manner suitable to resist the full wall anchorage design force and to carry that force across the diaphragm. In the direction parallel to the deck corrugations, subdiaphragm systems are considered near the walls; if the compression forces in the deck become large relative to the joist spacing, small compression reinforcing elements are provided to transfer the forces into the subdiaphragms.

C12.11.2.2.6 Eccentrically Loaded Anchorage System. Wall anchors often are loaded eccentrically, either because the anchorage mechanism allows eccentricity, or because of anchor bolt or strap misalignment. This eccentricity reduces the anchorage connection capacity and hence must be considered explicitly in design of the anchorage. Figure C12.11-2 shows a one-sided roof-to-wall anchor that is subjected to severe eccentricity due to a misplaced anchor rod. If the detail were designed as a concentric two-sided connection, this condition would be easier to correct.

C12.11.2.2.7 Walls with Pilasters. The anchorage force at pilasters must be calculated considering two-way bending in wall panels. It is customary to anchor the walls to the diaphragms assuming one-way bending and simple supports at the top and bottom of the wall. However, where pilasters are present in the walls, their stiffening effect must be taken into account. Each panel between pilasters is supported on four sides. The reaction at the pilaster top is the result of two-way action of the panel and is applied directly to the beam or girder anchorage at the top of the pilaster. The anchor load at the pilaster generally is larger than the typical uniformly distributed anchor load between pilasters. Figure C12.11-3 shows the tributary area typically used to determine the anchorage force for a pilaster.

Anchor points adjacent to the pilaster must be designed for the full tributary loading, conservatively ignoring the effect of the adjacent pilaster.

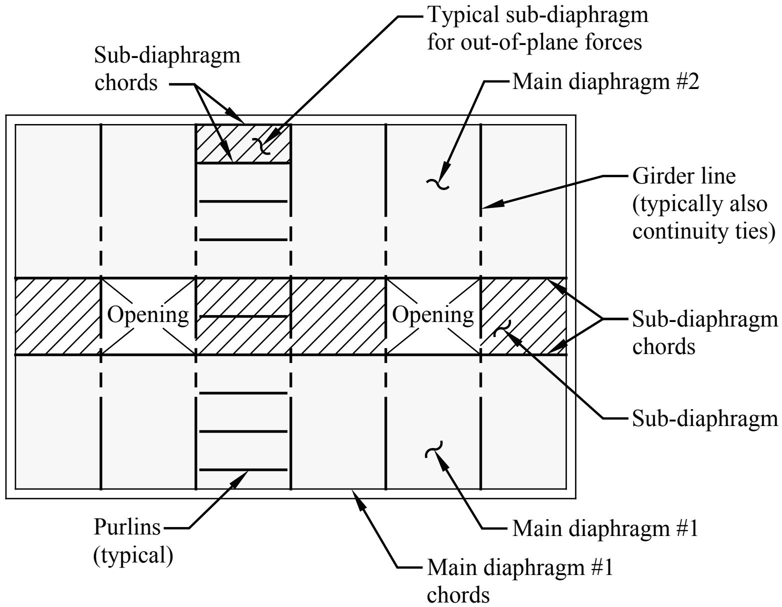


Figure C12.11-1 Typical subdiaphragm framing.

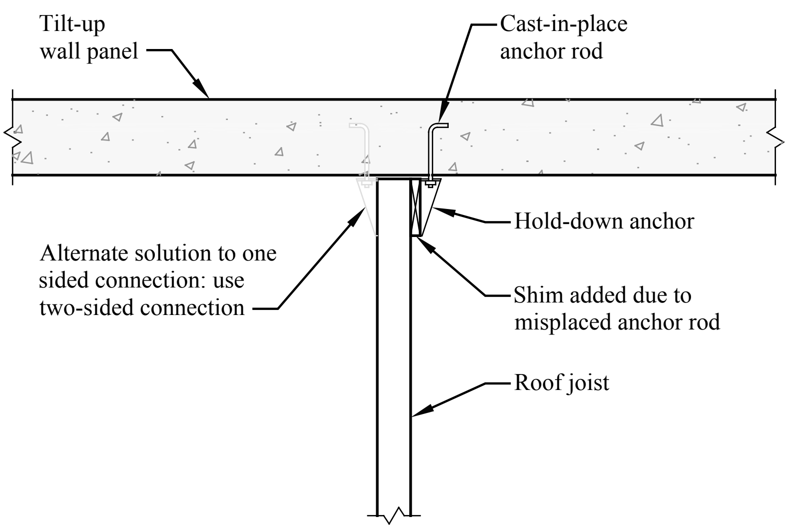


Figure C12.11-2 Plan view of wall anchor with misplaced anchor rod.

C12.12 DRIFT AND DEFORMATION

As used in the standard, deflection is the absolute lateral displacement of any point in a structure relative to its base, and story drift is the difference in deflection across a story (i.e., the deflection of a floor relative to that of the floor below).

The drifts and deflections are checked for the design earthquake ground motion, which is two-thirds of the maximum considered earthquake (MCE) ground motion.

There are many reasons to control drift; the most significant are to address the structural performance concerns of member inelastic strain and system stability and to limit damage of nonstructural components, which can be life-threatening. Drifts provide a direct but imprecise measure of member strain and structural stability. Under small lateral deformations, secondary stresses due to the P-delta effect are normally within tolerable limits. (See Section C12.8.7.) The drift limits provide indirect control of structural performance.

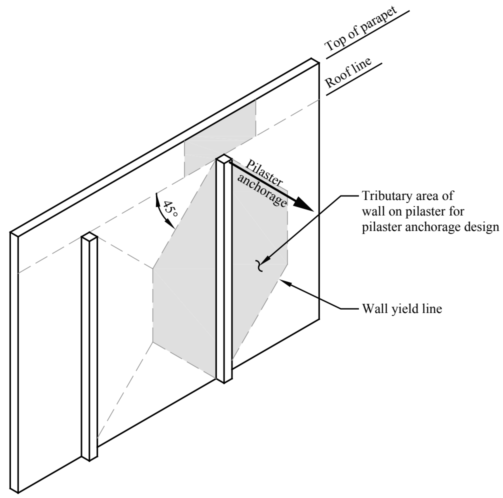


Figure 12.11-3 Tributary area used to determine anchorage force at pilaster.

Buildings subjected to earthquakes need drift control to restrict damage of partitions, shaft and stair enclosures, glass, and other fragile nonstructural elements. The drift limits have been established without regard to economic considerations such as a comparison of present worth of future repairs with additional structural costs to limit drift. These are matters for building owners and designers to address.

The drift limits of Table 12.12-1 reflect consensus judgment taking into account life safety and damage control objectives described above. Since the displacements induced in a structure include inelastic effects, structural damage in the design-level earthquake is likely. This may be seen from the seismic drift limits stated in Table 12.12-1. For ordinary structures (Occupancy Category I or II), the drift limit is $0.02h_{sx}$, which is about ten times the drift ordinarily allowed under wind loads. If deformations well in excess of the seismic drift limits were to occur repeatedly, structural components could lose so much stiffness or strength that they compromise the safety and stability of the structure.

To provide better performance for Occupancy Category IV essential facilities, their drift limits generally are more stringent than those for Occupancy Categories II and III. However, those limits are still greater than the damage thresholds for most nonstructural components. Therefore, while the performance of Occupancy Category IV buildings should be better than that of lower Occupancy Category buildings, there still can be considerable damage in the design earthquake.

The drift limits for low-rise structures are relaxed somewhat, provided that the interior walls, partitions, ceilings, and exterior wall systems have been designed to accommodate story drifts. The type of steel building envisioned by the exception to the table would be similar to a prefabricated steel structure with metal skin.

The limits set forth in Table 12.12-1 are for story drifts and apply to each and every story. For some structures, satisfying strength requirements may produce a system with adequate drift control. However, the design of moment-resisting frames and of tall, narrow shear walls or braced frames often is governed by drift considerations. Where design spectral response accelerations are large, seismic drift considerations are expected to control the design of midrise buildings. Where design spectral response accelerations are small or the building is very tall, design for wind generally will control.

C12.12.3 Building Separation. The intent of this section is to address separations (also called seismic joints) between adjacent structures or portions of the same structure (with or without frangible closures) for the purpose of permitting independent response to earthquake ground motion. For irregular structures that cannot be expected to act reliably as a unit, seismic joints should be used to produce separate units whose independent response to earthquake ground motion can be predicted.

The standard does not give a precise formulation for the separations, but it does require that the distance be "sufficient to avoid damaging contact under total deflection." It is recommended that the distance be no less than the square root of the sum of the squares of the lateral deflections, which represent the anticipated maximum inelastic deformations including torsion, of the two units assumed to deflect toward each other (thus increasing with height). If the effects of impact can be shown not to be detrimental, these distances can be reduced. For very rigid shear wall structures with rigid diaphragms whose lateral deflections cannot be reasonably estimated, it is suggested that older code requirements for structural separations of at least 1 inch (25 mm) plus 1/2 inch (13 mm) for each 10 feet (3 m) of height above 20 feet (6 m) be followed.

C12.12.4 Deformation Compatibility For Seismic Design Categories D Through F. The purpose of this section is to require that the seismic-force-resisting system provide adequate deformation control to protect elements of the structure that are not part of the seismic-force-resisting system. In regions of high seismicity, many designers apply ductile detailing requirements to elements that are intended to resist seismic forces but neglect such practices in nonstructural elements or elements intended to resist only gravity forces. Even where elements of the structure are not intended to resist seismic forces and are not detailed for such resistance, they can participate in the response and suffer severe damage as a result.

In the 1994 Northridge earthquake, such participation was a cause of several failures. A preliminary reconnaissance report of that earthquake (EERI, 1994) states:

> Of much significance is the observation that six of the seven partial collapses (in modern precast concrete parking structures) seem to have been precipitated by damage to the gravity load system. Possibly, the combination of large lateral deformation and vertical load caused crushing in poorly confined columns that were not detailed to be part of the lateral load resisting system. . . . Punching shear failures were observed in some structures at slab-to-column connections such as at the Four Seasons building in Sherman Oaks. The primary lateral load resisting system was a perimeter ductile frame that performed quite well. However, the interior slab-column system was incapable of undergoing the same lateral deflections and experienced punching failures.

This section addresses such concerns. Rather than relying on designers to assume appropriate levels of stiffness, this section explicitly requires that the stiffening effects of adjoining rigid structural and nonstructural elements be considered and that a rational value of member and restraint stiffness be used for the design of components that are not part of the seismic-force-resisting system.

This section also includes a requirement to address shears that can be induced in structural components that are not part of the seismic-force-resisting system, since sudden shear failures have been catastrophic in past earthquakes.

The exception in Section 12.12.4 is intended to encourage the use of intermediate or special detailing in beams and columns that are not part of the seismic-force-resisting system. In return for better detailing, such beams and columns are permitted to be designed to resist moments and shears from unamplified deflections. This reflects observations and experimental evidence that well-detailed components can accommodate large drifts by responding inelastically without losing significant vertical load-carrying capacity.

C12.13 FOUNDATION DESIGN

C12.13.3 Foundation Load-Deformation Characteristics. This section of the standard provides guidance on modeling load-deformation characteristics of the foundation-soil system (foundation stiffness) for linear analysis procedures. The further guidance contained herein addresses both linear and nonlinear analysis methods. Where linear analysis procedures are used with the methodology given below, the earthquake forces should not be reduced by R.

Modeling of the load-deformation characteristics of foundations should be in accordance with ASCE/SEI 41. For nonlinear analysis of piles that may form plastic hinges, the lateral load-deformation characteristics of piles may be taken from Song, et al. (2005).

For load combinations including seismic load effects, the vertical, lateral, and rocking load capacities of foundations as limited by the soil should be sufficient to resist loads with acceptable deformations, considering the short duration of loading, the dynamic properties of the soil, and the ultimate load capacities, Q_{us}, of the foundations.

Ultimate foundation load capacities should be determined by a qualified geotechnical engineer based on geotechnical site investigations that include field and laboratory testing to determine soil classification and soil strength parameters or on in-situ testing of prototype foundations. For competent soils that do not undergo strength degradation under seismic loading, strength parameters for static loading conditions may be used to compute ultimate load capacities for seismic design. For sensitive cohesive soils or saturated cohesionless soils, the potential for earthquake-induced strength degradation should be considered.

Ultimate foundation load capacities, Q_{us}, under vertical, lateral, and rocking loading should be determined using accepted foundation design procedures and principles of plastic analysis. Calculated ultimate load capacities, Q_{us}, should be best-estimated values using soil properties that are representative average values for individual foundations. Best-estimated values of Q_{us} should be reduced by resistance factors (ϕ) to reflect uncertainties in site conditions and in the reliability of analysis methods. The factored foundation load capacity, ϕQ_{us}, should be used both to check acceptance criteria and as the foundation capacity in nonlinear load-deformation models.

If ultimate foundation load capacities are determined based on geotechnical site investigations including laboratory or in-situ tests, ϕ factors equal to 0.8 for cohesive soils and 0.7 for cohesionless soils should be used for vertical, lateral, and rocking resistance for all foundation types. If ultimate foundation load capacities are determined based on full-scale field-testing of prototype foundations, ϕ factors equal to 1.0 for cohesive soils and 0.9 for cohesionless soils are recommended.

For both linear and nonlinear analysis procedures, a model incorporating a combined superstructure and foundation system is necessary to assess the effect of foundation deformations on the superstructure elements.

For linear analysis methods, the linear load-deformation behavior of foundations should be represented by an equivalent linear (secant) stiffness using soil properties that are compatible with the soil strain levels associated with the design earthquake motion. The strain-compatible shear modulus, G, and the associated strain-compatible shear wave velocity, v_s, needed for the evaluation of equivalent linear stiffness are specified in Chapter 19 of the standard or can be based on a site-specific study. ASCE/SEI 41 is an acceptable alternative to that contained in the standard and may provide more realistic results.

For nonlinear analysis procedures, the nonlinear load-deformation behavior of the foundation-soil system may be represented by a bilinear or multilinear curve having an initial equivalent linear stiffness and a limiting foundation capacity. The initial equivalent linear stiffness should be determined as described above for linear analysis methods. The limiting foundation capacity should be taken as the factored foundation load capacity, ϕQ_{us}. Parametric variations in analyses should include: (a) a reduction in stiffness of 50 percent combined with a limiting foundation capacity, ϕQ_{us}, and (b) an increase in stiffness of 50 percent combined with a limiting foundation capacity equal to Q_{us} multiplied by $1/\phi$.

For linear analysis procedures, factored foundation load capacities, ϕQ_{us}, should not be exceeded for load combinations that include seismic load effects.

For the nonlinear analysis procedures, if the factored foundation load capacity, ϕQ_{us}, is reached during seismic loading, the potential significance of associated transient and permanent foundation displacements should be evaluated. Foundation displacements are acceptable if they do not impair the continuing function of Occupancy Category IV structures or the life safety of any structure.

C12.13.4 Reduction of Foundation Overturning. Since the vertical distribution of forces prescribed for use with the equivalent lateral force procedure is intended to envelope story shears, overturning moments are exaggerated. (See Section C12.13.3.) Such moments will be lower where multiple modes respond, so a 25 percent reduction is permitted for design (strength and stability) of the foundation using this procedure. This reduction is not permitted for inverted pendulum or cantilevered column type structures, which typically have a single mode of response.

Since the modal response spectrum analysis procedure more accurately reflects the actual distribution of shears and overturning moments, the permitted reduction is only 10 percent.

C12.13.5 Requirements for Structures Assigned to Seismic Design Category C.

C12.13.5.1 Pole-Type Structures. The high contact pressures that develop between pole and soil as a result of lateral loads make pole-type structures sensitive to earthquake motions. Bending in the poles and soil lateral capacity and deformation are key considerations in the design. For further discussion of pole-soil interaction, see Section C12.13.6.7.

C12.13.5.2 Foundation Ties. One important aspect of adequate seismic performance is that the foundation acts as a unit, not permitting one column or wall to move appreciably with respect to another. To attain this performance, it is common to provide ties between footings and pile caps. This is especially important where the use of deep foundations is driven by the existence of soft surface soils.

Multistory buildings often have major columns that run the full height of the building adjacent to smaller columns that support only one level; the calculated tie force is based on the heavier column load.

The standard permits alternate methods of tying foundations together. Lateral soil pressure on pile caps is not a recommended method because motion is imparted from soil to structure and during displacement under dynamic conditions.

C12.13.5.3 Pile Anchorage Requirements. The pile anchorage requirements are intended to prevent brittle failures of the connection to the pile cap under moderate ground motions. Moderate ground motions can result in pile tension forces or bending moments that could compromise shallow anchorage embedment. Loss of pile anchorage could result in increased structural displacements from rocking, overturning instability, and loss of shearing resistance at the ground surface. A concrete bond to a bare steel pile section usually is unreliable, but connection by means of deformed bars properly developed from the pile cap into concrete confined by a circular pile section is permitted.

C12.13.6 Requirements for Structures Assigned to Seismic Design Categories D through F.

C12.13.6.1 Pole-Type Structures. See Section C12.13.5.1.

C12.13.6.2 Foundation Ties. See Section C12.13.5.2. For Seismic Design Categories D through F, the requirement is extended to spread footings on soft soils.

C12.13.6.3 General Pile Design Requirements. Design of piles is based on the same *R* factor used in design of the superstructure; since inelastic behavior will result, piles should be designed with ductility similar to that of the superstructure. When strong ground motions occur, inertial structure pile-soil interaction may produce plastic hinging in piles near the bottom of the pile cap, and kinematic soil-pile interaction will result in bending moments and shearing forces throughout the length of the pile, being higher at interfaces between stiff and soft soil strata. These effects are particularly severe in soft soils and liquefiable soils so Section 14.2.3.2.1 requires special detailing in areas of concern.

The shears and curvatures in piles caused by inertial and kinematic interaction may exceed the bending capacity of conventionally designed piles, resulting in severe damage. Analysis techniques to evaluate pile bending are discussed by Margason and Holloway (1977) and Mylonakis (2001), and these effects on concrete piles are further discussed by Shepard (1983). For homogeneous, elastic media and assuming the pile follows the soil, the free-field curvature (soil strains without a pile present) can be estimated by dividing the peak ground acceleration by the square of the shear wave velocity of the soil; considerable judgment is necessary in using this simple relationship for a layered, inelastic profile with pile-soil interaction effects. Norris (1994) discusses methods to assess pile-soil interaction.

Where determining the extent of special detailing, the designer must consider variation in soil conditions and driven pile lengths, so that adequate ductility is provided at potential high curvature interfaces. Confinement of concrete piles to provide ductility and to maintain functionality of the confined core pile during and after the earthquake may be obtained by use of heavy spiral reinforcement or use of exterior steel liners.

C12.13.6.4 Batter Piles. Partially embedded batter piles have a history of poor performance in strong ground shaking, as shown by Gerwick and Fotinos (1992). Failure of battered piles has been attributed to design that neglect loading on the piles from ground deformation or that assumes that lateral loads are resisted by axial response of piles without regard to moments induced in the pile at the pile cap (Lam and Bertero, 1990). Because batter piles are considered to have limited ductility, they must be designed using the load combinations with overstrength. Moment-resisting connections between pile and pile cap must resolve the eccentricities inherent in batter pile configurations. This concept is illustrated clearly by EQE Engineering (1991).

C12.13.6.5 Pile Anchorage Requirements. Piles should be anchored to the pile cap to permit energy dissipating mechanisms, such as pile slip at the pile-soil interface, while maintaining a competent connection. This section of the standard sets forth a capacity design approach to achieve that objective. Anchorages occurring at pile cap corners and edges should be reinforced to preclude local failure of plain concrete sections due to pile shears, axial loads, and moments.

C12.13.6.6 Splices of Pile Segments. A capacity design approach, similar to that for pile anchorage, is applied to pile splices.

C12.13.6.7 Pile Soil Interaction. Short piles and long slender piles embedded in the earth behave differently when subject to lateral forces and displacements. The response of a long slender pile depends on its interaction with the soil considering the nonlinear response of the soil. Numerous design aid curves and computer programs are available for this type of analysis, which is necessary to obtain realistic pile moments, forces, and deflections and is common in practice (Ensoft, 2004). More sophisticated models, which also consider inelastic behavior of the pile itself, can be analyzed using general-purpose nonlinear analysis computer programs or closely approximated using the pile-soil limit state methodology and procedure given by Song, et al. (2005).

Short piles (with length-to-diameter ratios no more than 6) can be treated as a rigid body simplifying the analysis. A method assuming a rigid body and linear soil response for lateral bearing is given in the current building codes. A more accurate and comprehensive approach using this method is given in a study by Czerniak (1957).

C12.13.6.8 Pile Group Effects. The effects of groups of piles, where closely spaced, must be taken into account for vertical and horizontal response. As groups of closely spaced piles move laterally, failure zones for individual piles overlap, and horizontal strength and stiffness response of the pile-soil system is reduced. Reduction factors or "*p*-multipliers" are used to account for these groups of closely spaced piles. For a pile center-to-center spacing of three pile diameters, reduction factors of 0.6 for the leading pile row and 0.4 for the trailing pile rows are recommended by Rollins, et al. (1999). Computer programs are available to analyze group effects assuming a nonlinear soil and elastic piles (Ensoft, 2004a).

C12.14 SIMPLIFIED ALTERNATIVE STRUCTURAL DESIGN CRITERIA FOR SIMPLE BEARING WALL OR BUILDING FRAME SYSTEMS

C12.14.1 General. In recent years, engineers and building officials have become concerned that the seismic design requirements in codes and standards, while intended to make structures perform more reliably, have become so complex and difficult to understand and to implement that they may be counterproductive. Since the response of buildings to earthquake ground shaking is very complex (especially for irregular structural systems), realistically accounting for these effects can lead to complex requirements. There is a concern that the typical designers of small, simple structures, which may represent more than 90 percent of construction in the United States, have difficulty understanding and applying the general seismic requirements of the standard.

The simplified procedure presented in this section of the standard applies to low-rise, stiff structures. The procedure, which was refined and tested over a five-year period, was developed to be used for a defined set of structures deemed to be sufficiently regular in configuration to allow a reduction of prescriptive requirements. For some design elements, such as foundations and anchorage of nonstructural systems, other sections of the standard must be followed, as referenced within Section 12.14.

C12.14.1.1 Simplified Design Procedure. Reasons for the limitations of the simplified design procedure of Section 12.14 are as follows:

1. The procedure was developed to address adequate seismic performance for standard occupancies. Since it was not developed for higher levels of performance associated with Occupancy Category III and IV structures, no importance factor (I) is employed.
2. Site Class E and F soils require specialized procedures that are beyond the scope of the procedure.
3. The procedure was developed for stiff, low-rise buildings, where higher-mode effects are negligible.
4. Only stiff systems, where drift is not a controlling design criterion, may employ the procedure. Because of this limitation, drifts are not computed. The response modification coefficient, R, and the associated system limitations are consistent with those found in the general Chapter 12 requirements.
5. In order to achieve a balanced design and to achieve a reasonable level of redundancy, two lines of resistance are required in each of the two major axis directions. Because of this stipulation, no redundancy factor (ρ) is applied.
6. To reduce the potential for dominant torsional response, at least one line of resistance must be placed on each side of the center of mass.
7. Large overhangs for flexible diaphragm buildings can produce response that is inconsistent with the assumptions associated with the procedure.
8. A system that satisfies these layout and proportioning requirements avoids torsional irregularity, so calculation of accidental torsional moments is not required.
9. An essentially orthogonal orientation of lines of resistance effectively uncouples response along the two major axis directions, so orthogonal effects may be neglected.
10. Where the simplified design procedure is chosen, it must be used for the entire design, in both major axis directions.
11. Since in-plane and out-of-plane offsets generally create large diaphragm, collector, and discontinuous element demands that are not addressed by the procedure, these irregularities are prohibited.
12. Buildings that exhibit weak-story behavior violate the assumptions used to develop the procedure.

C12.14.3 Seismic Load Effects and Combinations. The seismic load effect and combination equations for the simplified design procedure are consistent with those for the general procedure, with one notable exception: the overstrength factor (corresponding to Ω_0 in the general procedure) is set at 2.5 for all systems as indicated in Section 12.14.3.2.1. Given the limited systems that can use the simplified design procedure, specifying unique overstrength factors was deemed unnecessary.

C12.14.7 Design and Detailing Requirements. The design and detailing requirements outlined in this section are similar to those for the general procedure. The few differences include the following:

1. Forces used to connect smaller portions of a structure to the remainder of the structures are taken as 0.20 times the short-period design spectral response acceleration, S_{DS}, rather than the general procedure value of 0.133 (Section 12.14.7.1).
2. Anchorage forces for concrete or masonry structural walls for structures with diaphragms that are not flexible are computed using the requirements for nonstructural walls.

C12.14.8 Simplified Lateral Force Analysis Procedure

C12.14.8.1 Seismic Base Shear. The seismic base shear in the simplified design procedure, as given by Equation 12.14-11, is a function of the short-period design spectral response acceleration, S_{DS}. The value for F in the base shear equation addresses changes in dynamic response for two- and three-story buildings. As in the general procedure (Section 12.8.1.3), S_{DS} may be computed for short, regular structures with S_S taken no greater than 1.5.

C12.14.8.2 Vertical Distribution. The seismic forces for multistory buildings are distributed vertically in proportion to the weight of the respective floor. Given the slightly amplified base shear for multi-story buildings, this assumption, along with the three-story height limit, produces results consistent with the more traditional triangular distribution without introducing that more sophisticated approach.

C12.14.8.5 Drift Limits and Building Separation. For the simplified design procedure, which is restricted to stiff wall and braced frame structures, drift need not be calculated. Where drifts are required (such as for structural separations and cladding design) a conservative drift value of 1 percent is specified.

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COMMENTARY CHAPTER 13, SEISMIC DESIGN REQUIREMENTS FOR NONSTRUCTURAL COMPONENTS

C13.1 GENERAL

Chapter 13 defines minimum design criteria for architectural, mechanical, electrical, and other nonstructural systems and components recognizing structure use, occupant load, the need for operational continuity, and the interrelation of structural and architectural, mechanical, electrical, and other nonstructural components. Nonstructural components are designed for design earthquake ground motions as defined in Section 11.2 and determined in Section 11.4.4 of the standard. In contrast to structures, which are implicitly designed for a low probability of collapse when subjected to the maximum considered earthquake (MCE) ground motions, there are no implicit performance goals associated with the MCE for nonstructural components. Performance goals associated with the design earthquake are discussed in Section C13.1.3.

Suspended or attached nonstructural components that could detach either in full or in part from the structure during an earthquake are referred to as falling hazards and may represent a serious threat to property and life safety. Critical attributes that influence the hazards posed by these components include their weight, their attachment to the structure, their failure or breakage characteristics (e.g., certain types of glass), and their location relative to occupied areas (e.g., over an entry or exit, a public walkway, an atrium, or a lower adjacent structure). Architectural components that pose potential falling hazards include parapets, cornices, canopies, marquees, glass, large ornamental elements (e.g., chandeliers), and building cladding. In addition, suspended mechanical and electrical components (e.g., mixing boxes, piping, and ductwork) may represent serious falling hazards. Figures C13.1-1 through C13.1-4 show damage to nonstructural components in past earthquakes.

Figure C13.1-1 Hospital imaging equipment that fell from overhead mounts.

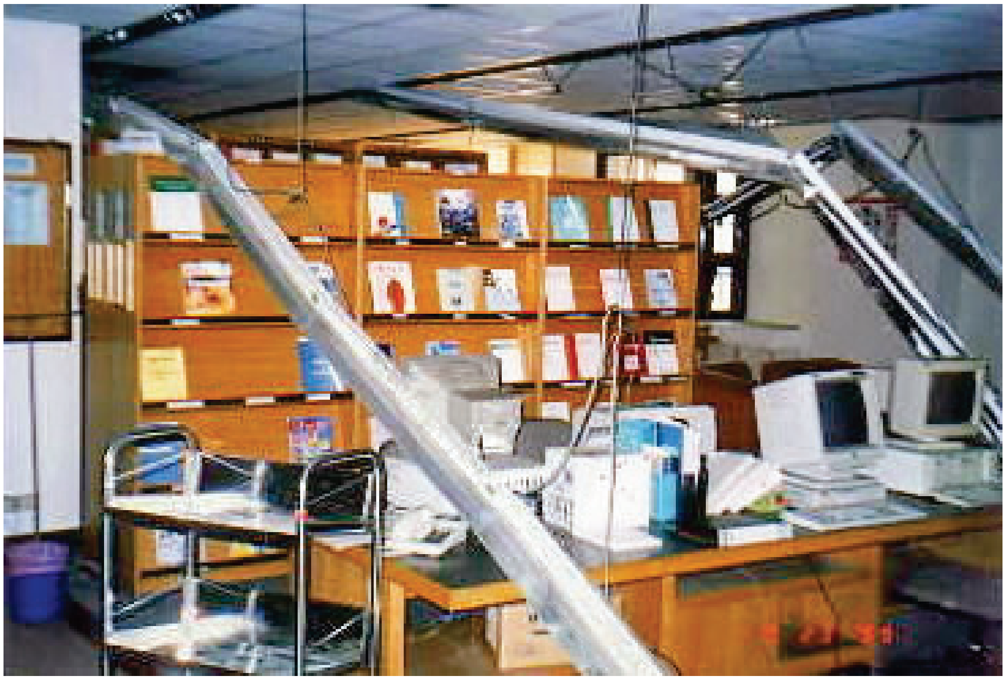

Figure C13.1-2 Collapsed light fixtures.

Figure C13.1-3 Collapsed duct and HVAC diffuser.

Figure C13.1-4 Damaged ceiling system.

Components whose collapse during an earthquake could result in blockage of the means of egress deserve special consideration. The term "means of egress" is used commonly in building codes with respect to fire hazard. Consideration of egress may include intervening aisles, doors, doorways, gates, corridors, exterior exit balconies, ramps, stairways, pressurized enclosures, horizontal exits, exit passageways, exit courts, and yards. Items whose failure could jeopardize the means of egress include walls around stairs and corridors, veneers, cornices, canopies, heavy partition systems, ceilings, architectural soffits, light fixtures, and other ornaments above building exits or near fire escapes. Examples of components that generally do not pose a significant falling hazard include fabric awnings and canopies. Architectural, mechanical, and electrical components that, if separated from the structure, will fall in areas that are not accessible to the public (e.g., into a mechanical shaft or light well) also pose little risk to egress routes.

For some architectural components such as exterior cladding elements, wind design forces may exceed the calculated seismic design forces. Nevertheless, seismic detailing requirements may still govern the overall structural design. Where this is a possibility, it must be investigated early in the structural design process.

The seismic design of nonstructural components may involve consideration of nonseismic requirements that are affected by seismic bracing. For example, accommodation of thermal expansion in pressure piping systems often is a critical design consideration and seismic bracing for these systems must be arranged in a manner that accommodates thermal movements. Particularly in the case of mechanical and electrical systems, the design for seismic loads should not compromise the functionality, durability, or safety of the overall design; this requires collaboration between the various disciplines of the design and construction team.

For various reasons (e.g., business continuity), it may be desirable to consider higher performance than that required by the building code. For example, to achieve continued operability of a piping system, it is necessary to prevent unintended operation of valves or other inline components in addition to preventing collapse and providing leak tightness. Higher performance also is required for components containing substantial quantities of hazardous contents (as defined in Section 11.2). These components must be designed to prevent uncontrolled release of those materials.

The requirements of Chapter 13 are intended to apply to new construction and tenant improvements installed at any time during the life of the structure, provided they are listed in Table 13.5-1 or 13.6-1. Further, they are intended to reduce (not eliminate) the risk to occupants and to improve the likelihood that essential facilities remain functional. While property protection (in the sense of investment preservation) is a possible consequence of implementation of the standard, it is not currently a stated or implied goal; a higher level of protection may be advisable if such protection is desired or required.

C13.1.1 Scope. The requirements for seismic design of nonstructural components apply to the nonstructural component as well as to its supports and attachments to the main structure. In some cases as defined in Section 13.2, it is necessary to consider explicitly the performance characteristics of the component. The requirements are intended to apply only to permanently attached components – not to furnishings, temporary items, or mobile units. Furnishings such as tables, chairs, and desks may shift during strong ground shaking but generally pose minimal hazards provided they do not obstruct emergency egress routes. Storage cabinets, tall bookshelves, and other items of significant mass do not fall into this category and should be anchored or braced in accordance with this chapter.

Figure C13.1-5 Toppled storage cabinets.

Temporary items are those that remain in place for short periods of time (months, not years). Components that, while movable or relocatable, are expected to remain in place for periods of a year or longer should be considered permanent for the purposes of this section. Modular office systems are considered permanent since they ordinarily remain in place for long periods. In addition, they often include storage units of significant capacity which may topple in an earthquake. They are subject to the provisions of Section 13.5.8 for partitions if they exceed 6 feet in height. Mobile units include components that are moved from one point in the structure to another during ordinary use. Examples include desktop computers, office equipment, and other components that are not permanently attached to the building utility systems (Figure C13.1-5). Components that are mounted on wheels to facilitate periodic maintenance or cleaning but that otherwise remain in the same location (e.g., server racks) are not considered moveable for the purposes of anchorage and bracing. Likewise, skid-mounted components (as shown in Figure C13.1-6) as well as the skids themselves are considered permanent equipment.

In all cases, equipment must be anchored if it is permanently attached to utility services (electricity, gas, and water). For the purposes of this requirement, "permanently attached" should be understood to include all electrical connections except NEMA 5-15 and 5-20 straight-blade connectors (duplex receptacles).

C13.1.2 Seismic Design Category. The requirements for nonstructural components are based in part on the Seismic Design Category to which they are assigned. As the Seismic Design Category is established considering factors not unique to specific nonstructural components, all nonstructural components occupying or attached to a structure are assigned to the same Seismic Design Category as the structure.

C13.1.3 Component Importance Factor. Performance expectations for nonstructural components often are defined in terms of the functional requirements of the structure to which the components are attached. While specific performance goals for nonstructural components have yet to be defined in building codes, the component importance factor (I_p) implies performance levels for specific cases. For noncritical nonstructural components (those with an importance factor, I_p, of 1.0) the following behaviors are anticipated for shaking having different levels of intensity:

1. Minor earthquake ground motions – minimal damage; not likely to affect functionality
2. Moderate earthquake ground motions – some damage that may affect functionality
3. Design earthquake ground motions – major damage but significant falling hazards are avoided; likely loss of functionality.

Figure C13.1-6 Skid-mounted components.

Components with importance factors greater than 1.0 are expected to remain in place, sustain limited damage, and, when necessary, function following an earthquake (see Section C13.2.2). These components can be located in structures that are not assigned to Occupancy Category IV. For example, fire sprinkler piping systems have an importance factor, I_p, of 1.5 in all structures since these essential systems should function following an earthquake.

The component importance factor is intended to represent the greater of the life-safety importance of the component and the hazard-exposure importance of the structure. It indirectly influences the survivability of the component via required design forces and displacement levels as well as component attachments and detailing. While this approach provides some degree of confidence in the seismic performance of a component, it may not be sufficient in all cases. For example, individual ceiling tiles may fall from a ceiling grid that has been designed for larger forces. This may not represent a serious falling hazard if the ceiling tiles are made of lightweight materials, but it may lead to blockage of critical egress paths or disruption of the facility function. When higher levels of confidence in performance are required, the component is classified as a designated seismic system (Section 11.2), and, in certain cases, seismic qualification of the component or system is necessary. Seismic qualification approaches are provided in Sections 13.2.5 and 13.2.6. In addition, seismic qualification approaches presently in use by the Department of Energy (DOE) can be applied.

Occupancy Category IV structures are intended to be functional following a design earthquake; critical nonstructural components and equipment in such structures are designed with I_p equal to 1.5. This requirement applies to most components and equipment since damage to vulnerable unbraced systems or equipment may disrupt operations following an earthquake even if they are not directly classified as essential to life safety. The nonessential/nonhazardous components themselves are not assessed, and requirements focus solely on the supports and attachments. UFC 3-310-04 has additional guidance for improved performance.

C13.1.4 Exemptions. Some nonstructural components either possess inherent strength and stability, are subject to low-level earthquake demands (accelerations and relative displacements), or both. Since these nonstructural components and systems are expected to achieve the performance goals described earlier in this commentary without explicitly satisfying additional requirements, they are exempt from the requirements of Chapter 13.

Chapter 13 does not apply to Seismic Design Category A due to its very low level of seismic hazard. (See Section C11.7.) With the exception of parapets supported by bearing walls or shear walls, all components in Seismic Design Category B are exempt due to the minimal level of seismic risk. Parapets are not exempt because experience has shown these items can fail and pose a significant falling hazard even at low shaking levels.

Mechanical and electrical components in Seismic Design Category C with an importance factor (I_p) equal to 1.0 are exempt because they are subject to low levels of seismic hazard, they do not contain hazardous substances, and their function is not required to maintain life safety following an earthquake. Small components with I_p equal to 1.0 in Seismic Design Categories D, E, and F also are exempt since they do not contain hazardous substances and are not large enough to pose a life-safety hazard if they fall, slide, or topple. Failures of unbraced distribution systems at or near the point of connection to nonstructural components have been observed in past earthquakes. For this reason, flexible connections such as expansion loops, braided hose, or expansion joints are required to allow for the larger relative displacements associated with unbraced components. Note that the stiffness of flexible connections may be sensitive to internal pressure and length of the connection.

C13.1.5 Applicability of Nonstructural Component and Requirements. At times, a nonstructural component should be treated as a nonbuilding structure. When the physical characteristics associated with a given class of nonstructural components vary widely, judgment is needed to select the appropriate design procedure and coefficients. For example, cooling towers vary from small packaged units with an operating weight of 2,000 pounds or less to structures the size of buildings. Consequently, design coefficients for the design of "cooling towers" are found both in Table 13.6-1 and Table 15.4-2. Small cooling towers are best designed as nonstructural components using the provisions of Chapter 13 while large ones are clearly nonbuilding structures that are more appropriately designed using the provisions of Chapter 15. Similar issues arise for other classes of nonstructural component (e.g., boilers and bins). Guidance on determining whether an item should be treated as a nonbuilding structure or nonstructural component for the purpose of seismic design is provided in Bachman and Dowty (2008).

There are practical limits on the size of a component that can be qualified via shake table testing. Components too large to be qualified by shake table testing need to be qualified by a combination of structural analysis and qualification testing or empirical evaluation through a subsystem approach. Subsystems of a large, complex component (e.g., a very large chiller) can be qualified individually and the overall structural frame of the component evaluated by structural analysis

Premanufactured modular mechanical units are considered nonbuilding structures supporting nonstructural components. The entire system (all modules assembled) can be shake table qualified or qualified separately as subsystems. Modular mechanical units house various nonstructural components that are subject to all the design requirements of Chapter 13.

The specified weight limit for nonstructural components (25 percent relative to the combined weight of the structure and component) relates to the condition at which dynamic interaction between the component and the supporting structural system is potentially significant. Section 15.3.2 contains requirements for addressing this interaction in design.

C13.1.6 Reference Documents. Professional and trade organizations have developed nationally recognized codes and standards for the design and construction of specific mechanical and electrical components. These documents provide design guidance for normal and upset (abnormal) operating conditions and for various environmental conditions. Some of these documents include earthquake design requirements in the context of the overall mechanical or electrical design. It is the intent of the standard that seismic requirements in referenced documents be used. The developers of these documents are familiar with the expected performance and failure modes of the components; however, the documents may be based on design considerations not immediately obvious to a structural design professional. For example, in the design of industrial piping, stresses due to seismic inertia forces typically are not added to those due to thermal expansion.

There is a potential for misunderstanding and misapplication of reference documents for the design of mechanical and electrical systems. A registered design professional familiar with both the standard and the reference documents used should be involved in the review and acceptance of the seismic design.

Even when reference documents for nonstructural components lack specific earthquake design requirements, mechanical and electrical equipment constructed in accordance with industry-standard reference documents have performed well historically when properly anchored. Nevertheless, it is expected that manufacturers of mechanical and electrical equipment will consider seismic loads in the design of the equipment itself even when not explicitly required by this chapter.

While some reference documents provide requirements for seismic capacity appropriate to the component being designed, the seismic demands used in design may not be less than those specified in the standard.

Specific guidance for selected mechanical and electrical components and conditions is provided in Section 13.6.

C13.1.7 Reference Documents Using Allowable Stress Design. Many nonstructural components are designed using specifically developed reference documents that are based on allowable stress loads and load combinations and permit increases in allowable stresses for seismic loading. Although Section 2.4.1 of the standard does not permit increases in allowable stresses, Section 13.1.7 explicitly defines the conditions for their use in the design of nonstructural components.

C13.2 GENERAL DESIGN REQUIREMENTS

C13.2.1 Applicable Requirements for Architectural, Mechanical, and Electrical Components, Supports, and Attachments. Compliance with the requirements of Chapter 13 may be accomplished by project-specific design or by a manufacturer's certification of seismic qualification of a system or component. In each case, the evidence of compliance is submitted to the authority having jurisdiction. When compliance is by manufacturer's certification, the items must be installed in accordance with the manufacturer's requirements.

Components addressed by the standard include individual simple units and assemblies of simple units for which reference documents establish seismic analysis or qualification requirements. Also addressed by the standard are complex architectural, mechanical, and electrical systems for which reference documents either do not exist or exist for only elements of the system. In the design and analysis of both simple components and complex systems, the concepts of flexibility and ruggedness often can assist the designer in determining the necessity for analysis and, when analysis is necessary, the extent and methods by which seismic adequacy may be determined. These concepts are discussed in Section C13.6.1.

C13.2.2 Special Certification Requirements for Designated Seismic Systems. While the goal of design for most nonstructural components is to prevent detachment or toppling that would pose a hazard to life safety, designated seismic systems (with $I_p = 1.5$) are intended to meet higher performance goals. In some cases, failure of mechanical or electrical equipment itself poses a significant hazard. This section addresses the design and certification of designated seismic system components and their supports and attachments. The goals of this section are to improve survivability and achieve a high level of confidence in post-earthquake functionality, and they require additional considerations.

Examples of designated seismic systems include fire protection piping, uninterruptible power supplies for hospitals, and certain vessels or piping that contain highly toxic or explosive substances.

Using an importance factor, I_p, equal to 1.5 to increase design forces can reduce the possibility of detachment or toppling, but this directly affects only structural integrity and stability; function and operability of mechanical and electrical components may be affected only indirectly by increasing design forces. For complex components, testing or experience may be the only reasonable way to improve the confidence of function and operability. For many types of equipment, past earthquake experience has shown that maintaining structural integrity and stability provides post-earthquake function and operability. On the other hand, mechanical joints in containment components (e.g., tanks, vessels, and piping) may not remain leak-tight in an earthquake. Avoiding this condition may require a design that is more conservative than that required by the standard.

Evaluating post-earthquake operational performance by analysis is impractical for active mechanical and electrical equipment with components that rotate or otherwise move mechanically during operation. Active equipment includes pumps and electric motors. In many cases, such equipment is inherently rugged, and an evaluation of experience data together with analysis of the component anchorage is adequate to demonstrate compliance (see Section 13.6). In other cases (e.g., motor control centers and switching equipment), shake table testing may be required. Components that contain hazardous materials (e.g., tanks, piping, and vessels) typically are qualified by analysis, but it may be necessary to qualify certain operational valving or mechanical equipment within the system by other means.

C13.2.3 Consequential Damage. Although the components identified in Tables 13.5-1 and 13.6-1 are listed separately, significant interrelationships exist and must be considered. Consequential damage occurs due to interaction between components and systems. Even "braced" components displace and the displacement between lateral supports can be significant in the case of distributed systems such as piping systems, cable and conduit systems and other linear systems. It is the intent of the standard that the seismic displacements considered include both relative displacement between multiple points of support (addressed in Section 13.3.2) and, for mechanical and electrical components, displacement within the component assemblies. Impact of components must be avoided unless the components are fabricated of ductile materials that have been shown to be capable of accommodating the expected impact loads. With protective coverings, ductile mechanical and electrical components and many more fragile components are expected to survive all but the most severe impact loads. Flexibility and ductility of the connections between distribution systems and the equipment to which they attach is essential to the seismic performance of the system.

The determination of the displacements that generate these interactions are not addressed explicitly in Section 13.3.2.1. That section concerns relative displacement of support points. Consequential damage may occur due to displacement of components and systems between support points. For example, in older suspended ceiling installations, excessive lateral displacement of a ceiling system may fracture sprinkler heads that project through the ceiling. A similar situation may arise if sprinkler heads projecting from a small diameter branch line pass through a rigid ceiling system. While the branch line may be properly restrained, it may still displace sufficiently between lateral support points, to impact other components or systems. Similar interactions occur where a relatively flexible distributed system connects to a braced or rigid component.

The potential for impact between components that are in contact with or in close proximity to other structural or nonstructural components must be considered. However, where considering these potential interactions, the designer must determine if the potential interaction is both credible and significant. For example, the fall of a ceiling panel located above a motor control center is a credible interaction because the falling panel in older suspended ceiling installations can reach and impact the motor control center. An interaction is significant if it can result in damage to the target. Impact of a ceiling panel on a motor control center may not be significant, due to the light weight of the ceiling panel. Special design consideration is appropriate where the failure of a nonstructural element could adversely influence the performance of an adjacent critical nonstructural component, such as an emergency generator.

C13.2.4 Flexibility. In many cases, flexibility is more important than strength in the performance of distributed systems, such as piping and ductwork. A good understanding of the displacement demand on the system as well as its displacement capacity is required. Components or their supports and attachments must be flexible enough to accommodate the full range of expected differential movements; some localized inelasticity is permitted in accommodating the movements. Relative movements in all directions must be considered. For example, even a braced branch line of a piping system will displace, so it needs to be connected to other braced or rigid components in a manner that will accommodate the displacements without failure (see Figure C13.2-1). For another example, cladding units (such as precast concrete wall units) while often very rigid in-plane, if supported at more than one level, require connections capable of accommodating story drift. (See Section C13.5.3 for an illustration.)

If component analysis assumes rigid anchors or supports, the predicted loads and local stresses can be unrealistically large, so it may be necessary to consider anchor and/or support stiffness.

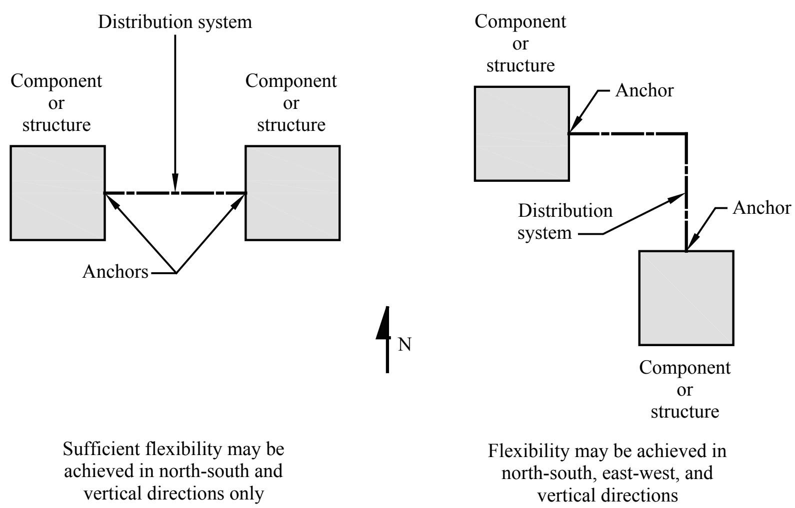

Figure C13.2-1 Schematic plans illustrating branch line flexibility.

C13.2.5 Testing Alternative for Seismic Capacity Determination. Testing is a well established alternative method of seismic qualification for small- to medium-size equipment. Several national reference documents have testing requirements adaptable for seismic qualification. One such reference document, ICC-ES AC156 (2007), is a shake-table testing protocol that has been adopted by the ICC Evaluation Service. It was developed specifically to be consistent with acceleration demands (that is, force requirements) of the standard.

The development or selection of testing and qualification protocols should at a minimum include the following:

1. Description of how the protocol meets the intent for the project-specific requirements and relevant interpretations of the standard.
2. Definition of a test input motion with a response spectrum that meets or exceeds the Design Earthquake spectrum for the site.
3. Accounting for dynamic amplification due to above-grade equipment installations. Consideration of the actual dynamic characteristics of the primary support structure is permitted, but not required.
4. Definition of how shake-table input demands were derived.
5. Definition and establishment of a verifiable pass/fail acceptance criterion for the seismic qualification based upon the equipment importance factor and consistent with the building code and project-specific design intent.

6. Development of criteria that can be used to rationalize test unit configuration requirements for highly variable equipment product lines.

To aid the design professional in assessing the adequacy of the manufacturer's certificate of compliance it is recommended that certificates of compliance include:

1. Product family or group covered
2. Building code(s) and standard(s) for which compliance was evaluated
3. Testing standard used
4. Performance objective and corresponding importance factor (I_p = 1.0 or I_p = 1.5)
5. Seismic demand for which the component is certified, including code and/or standard design parameters used to calculate seismic demand (such as values used for a_p, R_p, and site class)
6. Installation restrictions, if any (grade, floor, or roof level)

Without a test protocol recognized by the building code, qualification testing is inconsistent and difficult to verify. The use of ICC-ES AC156 simplifies the task of compliance verification since it was developed to address directly the testing alternative for nonstructural components, as specified in the standard. It also sets forth minimum test plan and report deliverables.

Use of other standards or ad-hoc protocols to verify compliance of nonstructural components with the requirement of the standard should be considered carefully and used only where project-specific requirements cannot be met otherwise.

Where other qualification test standards will be used, in whole or in part, it is necessary to verify compliance with this standard. For example, IEEE 693 indicates that it is to be used for the sole purpose of qualifying electrical equipment (specifically listed in the standard) for use in utility substations. Where equipment testing has been conducted to other standards (for instance, testing done in compliance with IEEE 693), a straightforward approach would be to permit evaluation, by the manufacturer, of the test plan and data to validate compliance with the requirements of ICC-ES AC156, because it was developed specifically to comply with the seismic demands of this standard.

The qualification of mechanical and electrical components for seismic loads alone may not be sufficient to achieve high performance objectives. Establishing a high confidence that performance goals will be met requires consideration of the performance of structures, systems (fluid, mechanical, electrical, instrumentation, etc.), and their interactions (for example interaction of seismic and other loads) as well as compliance with installation requirements.

C13.2.6 Experience Data Alternative for Seismic Capacity Determination. An established method of seismic qualification for certain types of nonstructural components is the assessment of data for the performance of similar components in past earthquakes. The seismic capacity of the component in question is extrapolated based on estimates of the demands (force, displacement) to which the components in the database were subjected. Procedures for such qualification have been developed for use in nuclear facility applications by the Seismic Qualification Utility Group (SQUG) of the Electric Power Research Institute.

The SQUG rules for implementing the use of experience data are described in a proprietary Generic Implementation Procedure (GIP) database. It is a collection of findings from detailed engineering studies by experts for equipment from a variety of utility and industrial facilities.

Valid use of experience data requires satisfaction of rules that address physical characteristics, manufacturer's classification and standards, and findings from testing, analysis, and expert consensus opinion.

Four criteria are used to establish seismic qualification by experience, as follows:

1. Seismic capacity versus demand (a comparison with a bounding spectrum)
2. Earthquake experience database cautions and inclusion rules
3. Evaluation of anchorage
4. Evaluation of seismic interaction

Experience data should be used with care, since the design and manufacture of components may have changed considerably in the intervening years. The use of this procedure is also limited by the relative rarity of strong motion instrument records associated with corresponding equipment experience data.

C13.2.7 Construction Documents. Where the standard requires seismic design of components or their supports and attachments, appropriate construction documents defining the required construction and installation must be prepared. This facilitates the special inspection and testing needed to provide a reasonable level of quality assurance. Of particular concern are large nonstructural components (such as rooftop chillers) whose manufacture and installation involves multiple trades and suppliers, and which impose significant loads on the supporting structure. In these cases, it is important that the construction documents used by the various trades and suppliers to satisfy the seismic design requirements are prepared by a registered design professional.

The information required to prepare construction documents for component installation includes the dimensions of the component, the locations of attachment points, the operating weight, and the location of the center of mass. For instance, if an anchorage angle will be attached to the side of a metal chassis, the gage and material of the chassis must be known so that the number and size of required fasteners can be determined. Or, when a piece of equipment has a base plate that will be anchored to a concrete slab with expansion anchors, the drawings must show the base plate's material and thickness, the diameter of the bolt holes in the plate, and the size and depth of embedment of the anchor bolts. If the plate will be elevated above the slab for leveling, the construction documents must also show the maximum gap permitted between the plate and the slab.

C13.3 SEISMIC DEMANDS ON NONSTRUCTURAL COMPONENTS

The seismic demands on nonstructural components, as defined in this section, are acceleration demands and relative displacement demands. Acceleration demands are represented by equivalent static forces. Relative displacement demands are provided directly and are based on either the actual displacements computed for the structure or the maximum allowable drifts that are permitted for the structure.

C13.3.1 Seismic Design Force. The seismic design force for a component depends on the weight of the component, the component importance factor, the component response modification factor, the component amplification factor, and the component acceleration at a point of attachment to the structure. The forces prescribed in this section of the standard reflect the dynamic and structural characteristics of nonstructural components. As a result of these characteristics, forces used for verification of component integrity and design of connections to the supporting structure typically are larger than those used for design of the overall seismic-force-resisting system.

Certain nonstructural components lack the desirable attributes of structures (such as ductility, toughness, and redundancy) that permit the use of greatly reduced lateral design forces. Thus values for the response modification factor, R_p, in Tables 13.5-1 and 13.6-1 generally are smaller than R values for structures. These R_p values, used to represent the energy absorption capability of a component and its attachments, depend on both overstrength and deformability. At present these potentially separate considerations are combined in a single factor. The tabulated values are based on the collective judgment of the responsible committee.

The 2005 edition of the standard includes significant adjustments to tabulated R_p values for certain mechanical and electrical systems. For example, the value of R_p for welded steel piping systems is increased from 3.5 to 9. The a_p value increased from 1.0 to 2.5, so while it might appear that forces on such piping systems have been reduced greatly, the net change is negligible, as R_p/a_p changes from 3.5 to 3.6. The minimum seismic design force of Equation 13.3-3, which governs in many cases, is unchanged.

The component amplification factor (a_p) represents the dynamic amplification of component responses as a function of the fundamental periods of the structure (T) and component (T_p). When components are designed or selected, the structural fundamental period is not always defined or readily available. The component fundamental period (T_p) is usually only accurately obtained by shake-table or pull-back tests and is not available for the majority of components. Tabulated a_p values are based on component behavior that is assumed to be either rigid or flexible. Where the fundamental period of the component is less than 0.06 seconds, dynamic amplification is not expected, and the component is considered rigid. The tabulation of assumed a_p values is not meant to preclude more precise determination of the component amplification factor where the fundamental periods of both structure and component are available. The NCEER formulation shown in Figure C13.3-1 may be used to compute a_p as a function of T_p/T.

Dynamic amplification occurs where the period of a nonstructural component closely matches that of any mode of the supporting structure, although this effect may not be significant depending on the ground motion. For most buildings, the primary mode of vibration in each direction will have the most influence on the dynamic amplification for nonstructural components. For long-period structures (such as tall buildings), where the period of vibration of the fundamental mode is greater than 3.5 times T_s, higher modes of vibration may have periods that more closely match the period of nonstructural components. For this case, it is recommended that amplification be considered using such higher mode periods in lieu of the higher fundamental period. This approach may be generalized by computing floor response spectra for various levels that reflect the dynamic characteristics of the supporting structure to determine how amplification will vary as a function of component period. Calculation of floor response spectra can be complex, but simplified procedures are presented in Kehoe and Hachem (2003). Consideration of nonlinear behavior of the structure greatly complicates the analysis.

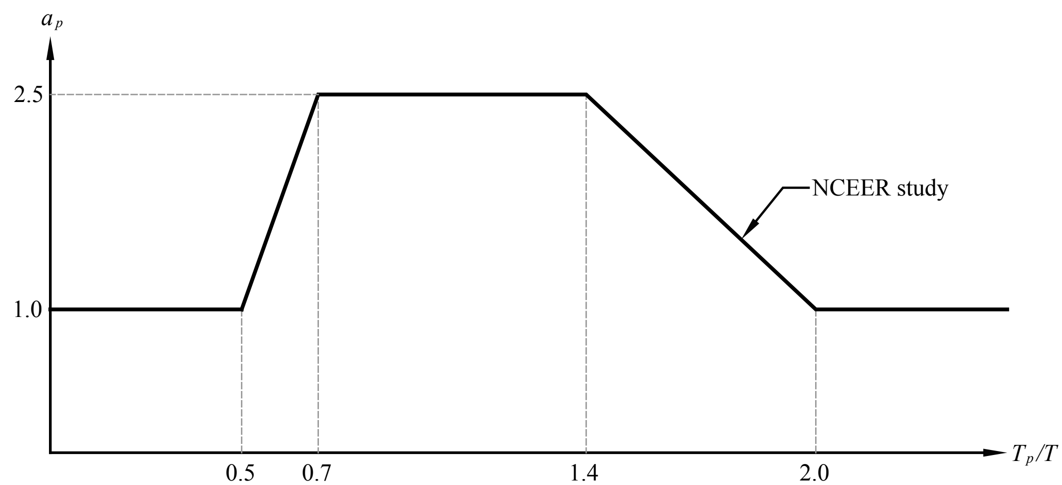

Figure C13.3-1 NCEER formulation for a_p as function of structural and component periods.

Equation 13.3-1 represents a trapezoidal distribution of floor accelerations within a structure, varying linearly from the acceleration at the ground (taken as $0.4S_{DS}$) to the acceleration at the roof (taken as $1.2S_{DS}$). The ground acceleration ($0.4S_{DS}$) is intended to be the same acceleration used as design input for the structure itself, including site effects. The roof acceleration is established as three times the input ground acceleration based on examination of recorded in-structure acceleration data for short and moderate height structures in response to large California earthquakes. Work by Miranda and Singh suggest that, for taller structures, the amplification with height may vary significantly due to higher mode effects. Where more information is available, Equation 13.3-4 permits an alternate determination of the component design forces based on the dynamic properties of the structure.

Equation 13.3-3 establishes a minimum seismic design force, F_p, that is consistent with current practice. Equation 13.3-2 provides a simple maximum value of F_p that prevents multiplication of the individual factors from producing a design force that would be unreasonably high, considering the expected nonlinear response of support and component. Figure C13.3-2 illustrates the distribution of the specified lateral design forces.

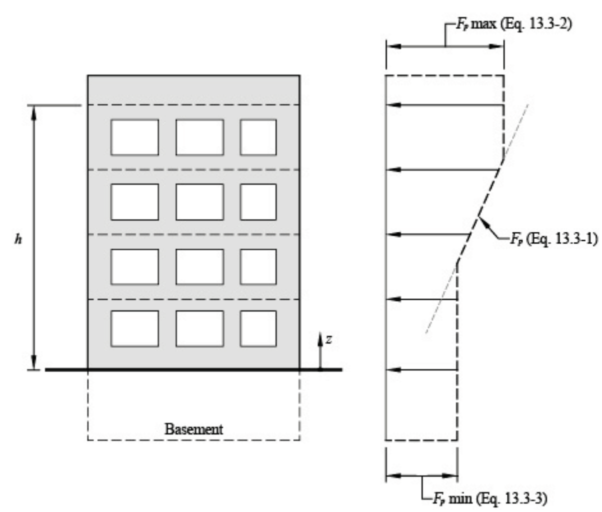

Figure C13.3-2 Lateral force magnitude over height.

For elements with points of attachment at more than one height, it is recommended that design be based on the average of values of F_p determined individually at each point of attachment (but with the entire component weight, W_p) using Equations 13.3-1 through 13.3-3.

Alternatively, for each point of attachment a force F_p may be determined using Equations 13.3-1 through 13.3-3, with the portion of the component weight, W_p, tributary to the point of attachment. For design of the component, the attachment force F_p must be distributed relative to the component's mass distribution over the area used to establish the tributary weight. To illustrate these options, consider a solid exterior nonstructural wall panel, supported top and bottom, for a one-story building with a rigid diaphragm. The values of F_p computed, respectively, for the top and bottom attachments using Equations 13.3-1 through 13.3-3 are $0.48S_{DS}I_pW_p$ and $0.30S_{DS}I_pW_p$. In the recommended method, a uniform load is applied to the entire panel based on $0.39S_{DS}I_pW_p$. In the alternative method, a trapezoidal load varying from $0.48S_{DS}I_pW_p$ at the top to $0.30S_{DS}I_pW_p$ at the bottom is applied. Each anchorage force is then determined considering static equilibrium of the complete component subject to all the distributed loads.

Cantilever parapets that are part of a continuous element should be checked separately for parapet forces. The seismic force on any component must be applied at the center of gravity of the component and must be assumed to act in any horizontal direction. Vertical forces on nonstructural components equal to $\pm 0.2S_{DS}W_p$ are specified in Section 13.3.1 and are intended to be applied to all nonstructural components and not just cantilevered elements. Nonstructural concrete or masonry walls laterally supported by flexible diaphragms must be anchored out-of-plane in accordance with Section 12.11.2.

C13.3.2 Seismic Relative Displacements. The equations of this section are for use in design of cladding, stairways, windows, piping systems, sprinkler components, and other components connected to one structure at multiple levels or to multiple structures. Two equations are given for each situation. Equations 13.3-5 and 13.3-7 produce structural displacements as determined by elastic analysis, unreduced by the structural response modification factor (R). Since the actual displacements may not be known when a component is designed or procured, Equations 13.3-6 and 13.3-8 provide upper-bound displacements based on structural drift limits. Use of upper-bound equations may facilitate timely design and procurement of components, but may also result in costly added conservatism.

The standard does not provide explicit acceptance criteria for the effects of seismic relative displacements, except for glazing. Damage to nonstructural components due to relative displacement is acceptable, provided the performance goals defined elsewhere in the chapter are achieved.

C13.3.2.1 Displacements within Structures. Seismic relative displacements can subject components or systems to unacceptable stresses. Nonstructural components designed with no intended structural function, such as infill walls, may interact with structural framing elements as a result of building deformation. The resulting stresses may exceed acceptable limits for the nonstructural components, the structural elements, or both. Consideration of this interrelationship is likely to govern the clearance between such components and the ductility and strength of their supports and attachments.

Where nonstructural components are supported between, rather than at, structural levels, as frequently occurs for glazing systems, partitions, stairs, veneers, and mechanical and electrical distributed systems, the height over which the displacement demand, D_p, must be accommodated may be less than the story height, h_{sx}, and should be considered carefully. For example, consider the glazing system supported by rigid precast concrete spandrels shown in Figure C13.3-3. The glazing system will be subjected to full story drift, D_p, although its height ($h_x - h_y$) is only a fraction of the story height. The design drift must be accommodated by anchorage of the glazing unit, the joint between the precast spandrel and the glazing unit, or some combination of the two. Similar displacement demands arise where pipes, ducts, or conduit that are braced to the floor or roof above are connected to the top of a tall, rigid, floor-mounted component.

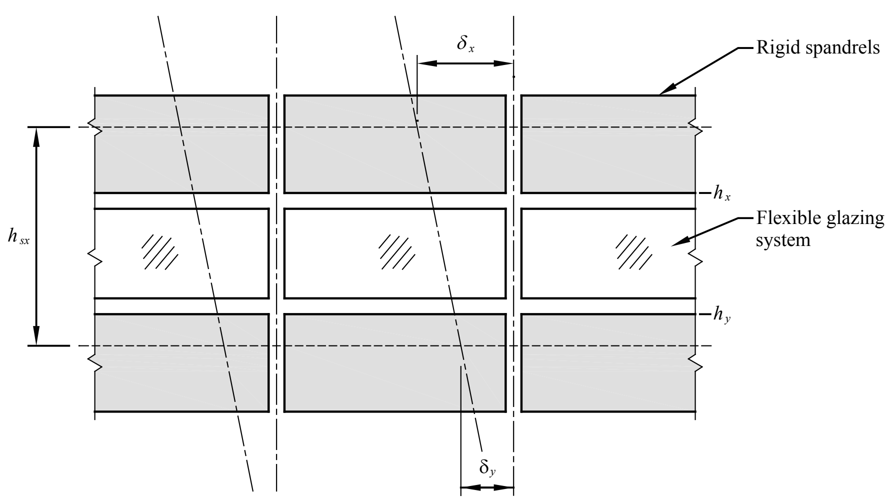

Figure C13.3-3 Displacements over less than story height.

For ductile components, such as steel piping fabricated with welded connections, the relative seismic displacements between support points can be more significant than inertial forces. Ductile piping can accommodate relative displacements by local yielding with strain accumulations well below failure levels. However, for components fabricated using less ductile materials, where local yielding must be avoided to prevent unacceptable failure consequences, relative displacements must be accommodated by flexible connections.

C13.3.2.2 Displacements between Structures. A component or system connected to two structures must accommodate horizontal movements in any direction, as illustrated in Figure C13.3-4.

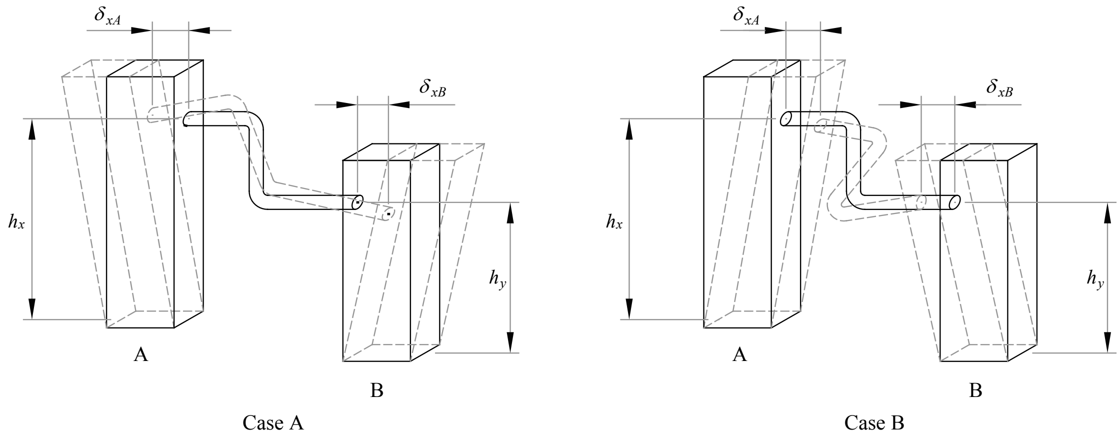

Figure C13.3-4 Displacements between structures.

C13.4 NONSTRUCTURAL COMPONENT ANCHORAGE

Unless exempted in Section 13.1.4, components must be anchored to the structure, and all required supports and attachments must be detailed in the construction documents. To satisfy the load path requirement of this section, the detailed information described in Section C13.2.7 must be communicated, during the design phase, to the registered design professional responsible for the design of the supporting structure.

Unanchored components often rock or slide when subjected to earthquake motions. Since this behavior may have serious consequences, is difficult to predict, and is exacerbated by vertical ground motions, positive restraint must be provided for each component.

The effective seismic weight used in design of the seismic force-resisting system must include the weight of supported components. To satisfy the load path requirements of this section, localized component demand must also be considered. This may be accomplished by checking the capacity of the first structural element in the load path (for example, a floor beam directly under a component) for combined dead, live, operating, and seismic loads, using the horizontal and vertical loads from Section 13.3.1 for the seismic demand, and repeating this procedure for each structural element or connection in the load path until the load case including horizontal and vertical loads from Section 13.3.1 no longer governs design of the element. The load path includes housekeeping slabs and curbs, which must be adequately reinforced and positively fastened to the supporting structure.

Since the exact magnitude and location of loads imposed on the structure may not be known until nonstructural components are ordered, the initial design of supporting structural elements should be based on conservative assumptions. The design of the supporting structural elements must be verified once the final magnitude and location of the design loads have been established.

Tests have shown there are consistent shear ductility variations between bolts installed in drilled or punched plates with nuts and connections using welded shear studs. The need for reductions in allowable loads for particular anchor types to account for loss of stiffness and strength may be determined through appropriate dynamic testing. Although comprehensive design recommendations are not available at present, this issue should be considered for critical connections subject to dynamic or seismic loading.

C13.4.2 Anchors in Concrete or Masonry. Design capacity for anchors in concrete must be determined in accordance with ACI 318 Appendix D. Design capacity for anchors in masonry is determined in accordance with ACI 530. Anchors must be designed to have ductile behavior or to provide a specified degree of excess strength. In either case, design forces are multiplied by 1.3 or based on the capacity of the component or its supports. The anchorage criteria provided in Chapter 13 specifically address the issue of non-ductile response and force amplification. Since the capacity of anchors in masonry is rarely governed by steel capacity, and failure in the masonry is non-ductile, an R_p of 1.5 should be used for design. Depending on the specifics of the design condition, ductile design of anchors in concrete may satisfy one or more of the following objectives:

1. Adequate load redistribution between anchors in a group
2. Allowance for anchor overload without brittle failure
3. Energy dissipation

Achieving deformable, energy-absorbing behavior in the anchor itself is often difficult. Unless the design specifically addresses the conditions influencing desirable hysteretic response (adequate gauge length, anchor spacing, edge distance, steel properties, etc.), anchors cannot be relied upon for energy dissipation. Simple geometric rules, such as restrictions on the ratio of anchor embedment length to depth, are not adequate to produce reliable ductile behavior. For example, a single anchor with sufficient embedment to force ductile tension failure in the steel body of the anchor bolt may still experience concrete fracture (a non-ductile failure mode) if the edge distance is small, the anchor is placed in a group of tension-loaded anchors with reduced spacing, or the anchor is loaded in shear instead of tension. In the common case where anchors are subject primarily to shear, response governed by the steel element may be non-ductile if the deformation of the anchor is constrained by rigid elements on either side of the joint. Designing the attachment so that its response is governed by a deformable link in the load path to the anchor is encouraged. This approach provides ductility and overstrength in the connection while protecting the anchor from overload. Ductile bolts should only be relied upon as the primary ductile mechanism of a system if the bolts are designed to have adequate gauge length (unbonded strained length of the bolt) to accommodate the anticipated nonlinear displacements of the system at the design earthquake. Guidance for determining the gauge length can be found in Part 3 of the *Provisions*.

Post-installed expansion and undercut anchors must be qualified in accordance with ACI 355.2-04, *Qualification of Post-Installed Mechanical Anchors in Concrete*. The ICC-ES acceptance criteria AC193 and AC308, which include specific provisions for screw anchors and adhesive anchors, also reference ACI 355.2. Reference to adhesives (such as in Section 13.5.7.2) apply, not to adhesive anchors, but to steel plates and other structural elements bonded or glued to the surface of another structural component with adhesive; such connections are generally non-ductile.

Anchors used to support towers, masts, and equipment are often provided with double nuts for leveling during installation. Where baseplate grout is specified at anchors with double nuts, it should not be relied upon to carry loads since it can shrink and crack or be omitted altogether. The design should include the corresponding tension, compression, shear, and flexure loads.

C13.4.3 Installation Conditions. Prying forces on anchors, which result from a lack of rotational stiffness in the connected part, can be critical for anchor design and must be considered explicitly.

For anchorage configurations that do not provide a direct mechanism to transfer compression loads (for example, a base plate that does not bear directly on a slab or deck but is supported on a threaded rod), the design for overturning must reflect the actual stiffness of baseplates, equipment, housing, and other elements in the load path when computing the location of the compression centroid and the distribution of uplift loads to the anchors.

C13.4.4 Multiple Attachments. While the standard does not prohibit the use of single anchor connections, it is good practice to use at least two anchors in any load-carrying connection whose failure might lead to collapse, partial collapse, or disruption of a critical load path.

C13.4.5 Power Actuated Fasteners. The capacity of power actuated fasteners in concrete often varies more than that of drilled post-installed anchors. The shallow embedment, small diameter, and friction mechanism of these fasteners make them particularly susceptible to the effects of concrete cracking. The suitability of power actuated fasteners to resist tension in concrete should be demonstrated by simulated seismic testing in cracked concrete.

Where properly installed in steel, power actuated fasteners typically exhibit reliable cyclic performance. Nevertheless, they should not be used singly to support suspended elements. Where used to attach cladding and metal decking, subassembly testing may be used to establish design capacities since the interaction between the decking, the sub-frame, and the fastener can only be estimated crudely by currently available analysis methods.

C13.4.6 Friction Clips. Friction clips, such as beam clamps, may loosen under cyclic loading, resulting in slippage or loss of connection capacity. Where friction clips are used, they may not be relied upon for seismic resistance. Fasteners that provide a positive mechanical connection have more reliable seismic performance. Clips that provide marginal mechanical connection, such as beam clamps that "dimple" the flange of the steel support may still rely chiefly on friction. These may not provide adequate cyclic capacity and should be qualified by seismic testing.

C13.5 ARCHITECTURAL COMPONENTS

For structures in Occupancy Category I through III, the requirements of Section 13.5 are intended to reduce property damage and life-safety hazards posed by architectural components due to loss of stability or integrity. When subjected to seismic motion, components may pose a direct falling hazard to building occupants or to people outside the building (as in the case of parapets, exterior cladding, and glazing). Failure or displacement of interior components (such as partitions and ceiling systems in exits and stairwells) may block egress.

For structures in Occupancy Category IV, the potential disruption of essential function due to component failure must also be considered.

Architectural component failures in earthquakes can be caused by deficient design or construction of the component, interrelationship with another component that fails, interaction with the structure, or inadequate attachment or anchorage. For architectural components, attachment and anchorage are typically the most critical concerns related to their seismic performance. Concerns regarding loss of function are most often associated with mechanical and electrical components. Architectural damage, unless very severe, can be accommodated temporarily. Very severe architectural damage is often accompanied by significant structural damage.

C13.5.1 General. Suspended architectural components are not required to satisfy the force and displacement requirements of Chapter 13, where prescriptive requirements are met. The requirements were relaxed in the 2005 edition of the standard to better reflect the consequences of the expected behavior. For example, impact of a suspended architectural ornament with a sheet metal duct may only dent the duct without causing a credible danger (assuming the ornament remains intact). The reference to Section 13.2.3 allows the designer to consider such consequences in establishing the design approach.

C13.5.2 Forces and Displacements. Partitions and interior and exterior glazing must accommodate story drift without failure that will cause a life-safety hazard. Design judgment must be used to assess potential life-safety hazards and the likelihood of life-threatening damage. Special detailing to accommodate drift for typical gypsum board or demountable partitions is unlikely to be cost-effective, and damage to these components poses a low hazard to life safety. Damage in these partitions occurs at low drift levels, but is inexpensive to repair.

If they must remain intact following strong ground motion, nonstructural fire-resistant enclosures and fire-rated partitions require special detailing that provides isolation from the adjacent or enclosing structure for deformation equivalent to the calculated drift (relative displacement). In-plane differential movement between structure and wall is permitted. Provision must be made for out-of-plane restraint. These requirements are particularly important in steel or concrete moment frame structures, which experience larger drifts. The problem is less likely to be encountered in stiff structures, such as those with shear walls.

Differential vertical movement between horizontal cantilevers in adjacent stories (such as cantilevered floor slabs) has occurred in past earthquakes. The possibility of such effects should be considered in the design of exterior walls.

C13.5.3 Exterior Nonstructural Wall Elements and Connections. Nonbearing wall panels that are attached to and enclose the structure must be designed to resist seismic (inertial) forces, wind forces, and gravity forces and to accommodate movements of the structure resulting from lateral forces and temperature change. The connections must allow wall panel movements due to thermal and moisture changes, and be designed so as to prevent the loss of load-carrying capacity in the event of significant yielding. Where wind loads govern, common practice is to design connectors and panels to allow for not less than two times the story drift caused by wind loads determined using a return period appropriate to the site location.

Design to accommodate seismic relative displacements often presents a greater challenge than design for strength. Story drifts can amount to 2 inches (50 mm) or more. Separations between adjacent panels are intended to limit contact and resulting panel misalignment or damage under all but extreme building response. Section 13.5.3(a) calls for a minimum separation of 1/2 inch (13 mm). For practical joint detailing and acceptable appearance, separations typically are limited to about 3/4 inch (19 mm). Manufacturing and construction tolerances for both wall elements and the supporting structure must be considered in establishing design joint dimensions and connection details.

Cladding elements, which are often very stiff in-plane, must be isolated so that they do not restrain and are not loaded by drift of the supporting structure. Slotted connections can provide isolation, but connections with long rods that flex achieve the desired behavior without requiring precise installation. Such rods must be designed to resist tension and compression in addition to induced flexural stresses, brittle, low-cycle fatigue failure.

Full-story wall panels are usually rigidly attached to and move with the floor structure nearest the panel bottom and isolated at the upper attachments. Panels also can be vertically supported at the top connections with isolation connections at the bottom. An advantage of this configuration is that failure of an isolation connection is less likely to result in complete detachment of the panel, since it will tend to rotate into the structure rather than away from it.

To minimize the effects of thermal movements and shrinkage on architectural cladding panels, connection systems are generally detailed to be statically determinate. Since the resulting support systems often lack redundancy, exacerbating the consequences of a single connection failure, fasteners must be designed for amplified forces and connecting members must be ductile. The intent is to keep inelastic behavior in the connecting members while the more brittle fasteners remain essentially elastic. To achieve this intent, the tabulated a_p and R_p values produce fastener design forces that are about 3 times those for the connecting members.

Limited deformability curtain walls, such as aluminum systems, are generally light and can undergo large deformations without separating from the structure. However, care must be taken in design of these elements so that low deformability components(as defined in Section 11.2) that may be part of the system, such as glazing panels, are detailed to accommodate the expected deformations without failure.

In Table 13.5-1, veneers are classified as either limited or low deformability elements. Veneers with limited deformability, such as vinyl siding, pose little risk. Veneers with low deformability, such as brick and ceramic tile, are highly sensitive to the performance of the supporting substrate. Significant distortion of the substrate results in veneer damage, possibly including separation from the structure. The resulting risk depends on the size and weight of fragments likely to be dislodged and on the height from which the fragments would fall. Detachment of large portions of the veneer can pose a significant risk to life. Such damage can be reduced by isolating veneer from displacements of the supporting structure. For structures with flexible lateral force-resisting systems, such as moment frames and buckling-restrained braced frames, approaches used to design non-bearing wall panels to accommodate story drift should be applied to veneers.

C13.5.5 Out-of-Plane Bending. The effects of out-of-plane application of seismic forces (defined in Section 13.3.1) on nonstructural walls, including the resulting deformations, must be considered. Where weak or brittle materials are employed, conventional deflection limits are expressed as a proportion of the span. The intent is to preclude out-of-plane failure of heavy materials (such as brick or block) or applied finishes (such as stone or tile).

C13.5.6 Suspended Ceilings. Suspended ceiling systems are fabricated using a wide range of building materials with differing characteristics. Some systems (such as lath and plaster or gypsum board, screwed or nailed to suspended members) are fairly homogeneous and should be designed as light-frame diaphragm assemblies, using the forces of Section 13.3 and the applicable material-specific design provisions of Chapter 14. Others comprise discrete elements laid into a suspension system and are the subject of this section.

Seismic performance of ceiling systems with lay-in or acoustical panels depends on support of the grid and individual panels at walls and expansion joints, integrity of the grid/panel assembly, interaction with other systems (such as fire sprinklers), and support for other nonstructural components (such as light fixtures and HVAC systems). Observed performance problems include dislodgement of tiles due to impact with walls and water damage (sometimes leading to loss of occupancy) due to interaction with fire sprinklers. Extensive shake table testing performed at the State University of New York at Buffalo addresses seismic performance of suspended ceiling systems at various ground motion levels. That work is reported by Yao (2000) and by Bidillo, et al. (2003, 2006, and 2007).

The performance of ceiling systems is affected by the placement of seismic bracing and the layout of light fixtures and other supported loads. Dynamic testing has demonstrated that splayed wires, even with vertical compression struts, may not adequately limit lateral motion of the ceiling system due to straightening of the end loops. Construction problems include slack installation or omission of bracing wires due to obstructions. Other testing has shown that unbraced systems may perform well where the system can accommodate the expected displacements, by providing both sufficient clearance at penetrations and wide closure members which are now required by the standard.

C13.5.6.1 Seismic Forces. Where the weight of the ceiling system is distributed non-uniformly, that condition should be considered in the design, since the typical T-bar ceiling grid has limited ability to redistribute lateral loads.

C13.5.6.2 Industry Standard Construction. Industry standard construction relies on ceiling contact with the perimeter wall for restraint. The key to good seismic performance is sufficiently wide closure angles at the perimeter to accommodate relative ceiling motion and adequate clearance at penetrating components (such as columns and piping) to avoid concentrating restraining loads on the ceiling system.

C13.5.6.2.1 Seismic Design Category C. While there is no direct equivalency between Seismic Design Categories and seismic zones, application of CISCA requirements for Seismic Zones 0 to 2 produces reasonable results for Seismic Design

Category C. ASTM E580 is currently being revised for consistency with the IBC and ASCE/SEI 7-05. When updated, it is expected to replace the CISCA requirements.

C13.5.6.2.2 Seismic Design Categories D through F. Where certain prescriptive requirements are met, lateral restraints may be omitted for small areas of suspended ceiling. The behavior of an unbraced ceiling system is similar to that of a pendulum; therefore, the lateral displacement is a function of the level of ground motion and the square root of the suspension length. The default displacement limit is based on anticipated damping and energy absorption of the suspended ceiling system without significant impact with the perimeter wall.

The requirements set forth in this section of the standard for Seismic Design Categories D through F are in addition to the CISCA requirements for Seismic Zones 3 and 4. Therefore, seismic requirements for ceilings are triggered where ceiling areas exceed 256 square feet, and additional requirements apply where ceiling areas exceed 1,000 square feet and 2,500 square feet. The alternative to provide swing joint connections or flexible devices (such as hoses) for sprinkler drops is included in the latest edition of NFPA 13.

C13.5.6.3 Integral Construction. Ceiling systems utilizing integral construction are constructed of modular pre-engineered components, which integrate lights, ventilation components, fire-sprinklers, and seismic bracing into a complete system. They may include aluminum, steel, and PVC components and may be designed using integral construction of ceiling and wall. They often use rigid grid and bracing systems, which provide lateral support for all the ceiling components, including sprinkler drops. This reduces the potential for adverse interactions between components, and eliminates the need to provide clearances for differential movement.

C13.5.7 Access Floors

C13.5.7.1 General. In past earthquakes and in cyclic load tests, some typical raised access floor systems behaved in a brittle manner and exhibited little reserve capacity beyond initial yielding or failure of critical connections. Testing shows that unrestrained individual floor panels may pop out of the supporting grid unless mechanically fastened to supporting pedestals or stringers. This may be a concern, particularly in egress pathways.

For systems with floor stringers, it is accepted practice to calculate the seismic force, F_p, for the entire access floor system within a partitioned space and then distribute the total force to the individual braces or pedestals. For stringerless systems, the seismic load path should be established explicitly.

Overturning effects subject individual pedestals to vertical loads well in excess of the weight, W_p, used in determining the seismic force, F_p. It is unconservative to use the design vertical load simultaneously with the design seismic force for design of anchor bolts, pedestal bending, and pedestal welds to base plates. “Slip on” heads that are not mechanically fastened to the pedestal shaft and thus cannot transfer tension are likely unable to transfer to the pedestal the overturning moments generated by equipment attached to adjacent floor panels.

To preclude brittle failure, each element in the seismic load path must have energy absorbing capacity. Buckling failure modes should be prevented. Lower seismic force demands are allowed for special access floors that are designed to preclude brittle and buckling failure modes.

C13.5.7.2 Special Access Floors. An access floor can be a “special access floor” if the registered design professional opts to comply with the requirements of Section 13.5.7.2. Special access floors include construction features that improve the performance and reliability of the floor system under seismic loading. The provisions focus on providing a reliable load path for seismic shear and overturning forces. Special access floors are designed for smaller lateral forces, and their use is encouraged at facilities with higher nonstructural performance objectives.

C13.5.8 Partitions. Partitions subject to these requirements must have independent lateral support bracing from the top of the partition to the building structure or to a substructure attached to the building structure. Some partitions are designed to span vertically from the floor to a suspended ceiling system. The ceiling system must be designed to provide lateral support for the top of the partition. An exception to this condition is provided to exempt bracing of light (gypsum board) partitions where the load does not exceed the minimum partition lateral load. Experience has shown that partitions subjected to the minimum load can be braced to the ceiling without failure.

C13.5.9 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions. The performance of glass in earthquakes falls into one of four categories:

1. The glass remains unbroken in its frame or anchorage.
2. The glass cracks but remains in its frame or anchorage while continuing to provide a weather barrier, and to be otherwise serviceable.
3. The glass shatters but remains in its frame or anchorage in a precarious condition, likely to fall out at any time.

4. The glass falls out of its frame or anchorage, either in shards or as whole panels.

Categories 1 and 2 satisfy both immediate-occupancy and life-safety performance objectives. Although the glass is cracked in Category 2, immediate replacement is not required. Categories 3 and 4 cannot provide for immediate occupancy, and their provision of life safety depends on the post-breakage characteristics of the glass and the height from which it can fall. Tempered glass shatters into multiple, pebble-size fragments that fall from the frame or anchorage in clusters. These broken glass clusters are relatively harmless to humans when they fall from limited heights, but they could be harmful when they fall from greater heights.

C13.5.9.1 General. Equation 13.5-1 is derived from *Earthquake Safety Design of Windows*, published in November 1982 by the Sheet Glass Association of Japan and is similar to an equation in Bouwkamp and Meehan (1960) that permits calculation of the story drift required to cause glass-to-frame contact in a given rectangular window frame. Both calculations are based on the principle that a rectangular window frame (specifically, one that is anchored mechanically to adjacent stories of a structure) becomes a parallelogram as a result of story drift, and that glass-to-frame contact occurs when the length of the shorter diagonal of the parallelogram is equal to the diagonal of the glass panel itself. The value $\Delta_{fallout}$ represents the displacement capacity of the system and D_p represents the displacement demand.

The 1.25 factor in the requirements described above reflect uncertainties associated with calculated inelastic seismic displacements of building structures. Wright (1989) states that “post-elastic deformations, calculated using the structural analysis process, may well underestimate the actual building deformation by up to 30 percent. It would therefore be reasonable to require the curtain wall glazing system to withstand 1.25 times the computed maximum interstory displacement to verify adequate performance.”

The reason for Exception 2 to Equation 13.5-1 is that the tempered glass, if shattered, would not produce an overhead falling hazard to adjacent pedestrians, although some pieces of glass may fall out of the frame.

C13.5.9.2 Seismic Drift Limits for Glass Components. As an alternative to the prescriptive approach of Section 13.5.9.1, the deformation capacity of glazed curtain wall systems may be established by test.

C13.6 MECHANIAL AND ELECTRICAL COMPONENTS

These requirements, focused on design of supports and attachments, are intended to reduce the hazard to life posed by loss of component structural stability or integrity. The requirements increase the reliability of component operation but do not address functionality directly. For critical components where operability is vital, Section 13.2.2 provides methods for seismically qualifying the component.

Traditionally, mechanical equipment without rotating or reciprocating components (such as tanks and heat exchangers) is anchored directly to the structure. Mechanical and electrical equipment with rotating or reciprocating components often is isolated from the structure by vibration isolators (such as rubber-in-shear, springs, or air cushions). Heavy mechanical equipment (such as large boilers) may not be restrained at all, and electrical equipment other than generators, which are normally isolated to dampen vibrations, usually is rigidly anchored (for example, switchgear and motor control centers).

Two distinct levels of earthquake safety are considered in the design of mechanical and electrical components. At the usual safety level, failure of the mechanical or electrical component itself due to seismic effects poses no significant hazard. In this case, design of the supports and attachments to the structure is required to avoid a life-safety hazard. At the higher safety level, the component must continue to function acceptably following the design earthquake. Such components are defined as designated seismic systems in Section 11.2 and may be required to meet the special certification requirements of Section 13.2.2.

Not all equipment or parts of equipment need to be designed for seismic forces. Where I_p is specified to be 1.0, damage to, or even failure of, a piece or part of a component does not violate these requirements as long as a life-safety hazard is not created. The restraint or containment of a falling, breaking, or toppling component (or its parts) by means of bumpers, braces, guys, wedges, shims, tethers, or gapped restraints to satisfy these requirements often is acceptable, although the component itself may suffer damage.

Judgment is required to fulfill the intent of these requirements; the key consideration is the threat to life safety. For example, a nonessential air handler package unit that is less than 4 feet (1.2 meters) tall bolted to a mechanical room floor is not a threat to life as long as it is prevented from significant displacement by having adequate anchorage. In this case, seismic design of the air handler itself is unnecessary. On the other hand, a 10-foot (3.0 meters) tall tank on 6-foot (1.8 meters) long angles used as legs, mounted on a roof near a building exit does pose a hazard. The intent of these requirements is that the supports and attachments (tank legs, connections between the roof and the legs, and connections between the legs and the tank), and possibly even the tank itself be designed to resist seismic forces. Alternatively, restraint of the tank by guys or bracing could be acceptable.

It is not the intent of the standard to require the seismic design of shafts, buckets, cranks, pistons, plungers, impellers, rotors, stators, bearings, switches, gears, nonpressure retaining casings and castings, or similar items. Where the potential for a hazard to life exists, it is expected that design effort will focus on equipment supports including base plates, anchorages, support lugs, legs, feet, saddles, skirts, hangers, braces, or ties.

Many mechanical and electrical components consist of complex assemblies of parts that are manufactured in an industrial process that produces similar or identical items. Such equipment may include manufacturer's catalog items and often are designed by empirical (trial-and-error) means for functional and transportation loadings. A characteristic of such equipment is that it may be inherently rugged. The term "rugged" refers to an ampleness of construction that provides such equipment with the ability to survive strong motions without significant loss of function. By examining such equipment, an experienced design professional usually should be able to confirm such ruggedness. The results of an assessment of equipment ruggedness may be used in determining an appropriate method and extent of seismic design or qualification effort.

C13.6.1 General. The exception allowing unbraced suspended components has been clarified, addressing concerns about the type of nonstructural components allowed by these exceptions as well as the acceptable consequences of interaction between components. In previous editions of the standard, certain nonstructural components that could represent a fire hazard following an earthquake were exempt from lateral bracing requirements. In the revised exception, reference to Section 13.2.3 addresses such concerns while distinguishing between credible seismic interactions and incidental interactions.

The seismic demand requirements are based on component structural attributes of flexibility (or rigidity) and ruggedness. Table 13.6-1 provides seismic coefficients based on judgments of the component flexibility, expressed in the a_p term, and ruggedness expressed in the R_p term. It may also be necessary to consider the flexibility and ductility of the attachment system that provides seismic restraint.

Entries for components and systems in Table 13.6-1 are grouped and described to improve clarity of application. Components are divided into three broad groups, within which they are further classified depending on the type of construction or expected seismic behavior. For example, mechanical components include "air-side" components (such as fans and air handlers) that experience dynamic amplification but are light and deformable; "wet-side" components that generally contain liquids (such as boilers and chillers) that are more rigid and somewhat ductile; and very rugged components (such as engines, turbines, and pumps) that are of massive construction due to demanding operating loads, and generally perform well in earthquakes, if adequately anchored.

A distinction is made between components isolated using neoprene and those that are spring isolated. Spring isolated are assigned a lower R_p value since they tend to have less effective damping. Internally isolated components are classified explicitly to avoid confusion.

C13.6.2 Component Period. Component period is used to clarify components as rigid ($T \leq 0.06$s) or flexible ($T > 0.06$s). Determination of the fundamental period of a mechanical or electrical component using analytical or test methods can become very involved. If not properly performed, the fundamental period may be underestimated, producing unconservative results. The flexibility of the component's supports and attachments typically dominates response and thus fundamental component period. Therefore, analytical determinations of component period must consider those sources of flexibility. Where determined by testing, the dominant mode of vibration of concern for seismic evaluation must be excited and captured by the test setup. This dominant mode of vibration cannot be discovered through in-situ tests that measure only ambient vibrations. To excite the mode of vibration with the highest fundamental period by in-situ tests, relatively significant input levels of motion are required (that is, the flexibility of the base and attachment must be exercised). A resonant frequency search procedure, such as that given in ICC-ES AC156, may be used to identify the dominant modes of vibration of a component.

Many types of mechanical components have fundamental periods below 0.06 seconds and may be considered to be rigid. Examples include horizontal pumps, engine generators, motor generators, air compressors, and motor driven centrifugal blowers. Other types of mechanical equipment are very stiff, but may have fundamental periods up to about 0.125 seconds. Examples include vertical immersion and deep well pumps, belt driven and vane axial fans, heaters, air handlers, chillers, boilers, heat exchangers, filters, and evaporators. These fundamental period estimates do not apply where the equipment is mounted on vibration isolators.

Electrical equipment cabinets can have fundamental periods of about 0.06 to 0.3 seconds, depending upon the supported weight and its distribution, the stiffness of the enclosure assembly, the flexibility of the enclosure base, and the load path through to the attachment points. Tall, narrow motor control centers and switchboards lie at the upper end of this period range. Low- and medium-voltage switchgear, transformers, battery chargers, inverters, instrumentation cabinets, and instrumentation racks usually have fundamental periods ranging from 0.1 to 0.2 seconds. Braced battery racks, stiffened vertical control panels, benchboards, electrical cabinets with top bracing, and wall-mounted panelboards have fundamental periods ranging from 0.06 to 0.1 seconds.

C13.6.3 Mechanical Components and C13.6.4 Electrical Components. Most mechanical and electrical equipment is inherently rugged and, where properly attached to the structure, has performed well in past earthquakes. Since the operational and transportation loads for which the equipment is designed typically are larger than those due to earthquakes, these requirements focus primarily on equipment anchorage and attachments. However, Designated Seismic Systems, which are required to function following an earthquake or which must maintain containment of flammable or hazardous materials, must themselves be designed for seismic forces or be qualified for seismic loading in accordance with Section 13.2.2.

The likelihood of post-earthquake operability can be increased where the following measures are taken:

1. Internal assemblies, subassemblies, and electrical contacts are attached sufficiently to prevent their being subjected to differential movement or impact with other internal assemblies or the equipment enclosure.
2. Operators, motors, generators, and other such components that are functionally attached to mechanical equipment by means of an operating shaft or mechanism are structurally connected or commonly supported with sufficient rigidity such that binding of the operating shaft will be avoided.
3. Any ceramic or other nonductile components in the seismic load path are specifically evaluated.
4. Adjacent electrical cabinets are bolted together and cabinet lineups are prevented from impacting adjacent structural members.

Components that could be damaged, or could damage other components, and are fastened to multiple locations of a structure must be designed to accommodate seismic relative displacements. Such components include bus ducts, cable trays, conduit, elevator guide rails, and piping systems. As discussed in Section C13.3.2.1, special design consideration is required where full story drift demands are concentrated in a fraction of the story height.

C13.6.5 Component Supports. The intent of this section is to require seismic design of all mechanical and electrical component supports to prevent sliding, falling, toppling, or other movement that could imperil life. Component supports are differentiated here from component attachments to emphasize that the supports themselves, as enumerated in the text, require seismic design even if fabricated by the mechanical or electrical component manufacturer. This is regardless of whether the mechanical or electrical component itself is designed for seismic loads.

C13.6.5.1 Design Basis. Standard supports are those developed in accordance with a reference document (Section 13.1.6). Where standard supports are not used, the seismic design forces and displacement demands of Chapter 13 are used with applicable material-specific design procedures of Chapter 14.

C13.6.5.2 Design for Relative Displacement. For some items, such as piping, seismic relative displacements between support points are of more significance than inertial forces. Components made of high deformability materials such as steel or copper can accommodate relative displacements inelastically, provided the connections also provide high deformability. Threaded and soldered connections exhibit poor ductility under inelastic displacements, even for ductile materials. Components made of less ductile materials can accommodate relative displacement effects only if appropriate flexibility or flexible connections are provided.

Detailing distribution systems that connect separate structures with bends and elbows makes them less prone to damage and less likely to fracture and fall, provided the supports can accommodate the imposed loads.

C13.6.5.3 Support Attachment to Component. As used in this Section, "integral" relates to the manufacturing process, not the location of installation. For example, both the legs of a cooling tower and the attachment of the legs to the body of the cooling tower must be designed, even if the legs are provided by the manufacturer and installed at the plant. Also, if the cooling tower has an I_p=1.5, the design must address not only the attachments (welds, bolts, etc.) of the legs to the component but also local stresses imposed on the body of the cooling tower by the support attachments.

C13.6.5.5 Additional Requirements. As reflected in this Section of the standard and in the footnote to Table 13.6-1, vibration isolated equipment with snubbers is subject to amplified loads as a result of dynamic impact.

Use of expansion anchors for non-vibration isolated mechanical equipment rated over 10 hp is prohibited based on experience with older anchor types. The ASCE/SEI 7 Seismic Subcommittee developing the 2010 edition of the standard is considering a proposal that would allow anchors qualified by simulated seismic testing and long-term vibration testing to also be exempt.

C13.6.6 Utility and Service Lines. For essential facilities (Occupancy Category IV), auxiliary on-site mechanical and electrical utility sources are recommended.

Where utility lines pass through the interface of adjacent, independent structures, they must be detailed to accommodate differential displacement computed in accordance with Section 13.3.2 and including the C_d factor of Section 12.2.1.

As specified in Section 13.1.3, nonessential piping whose failure could damage essential utilities in the event of pipe rupture are deemed Designated Seismic Systems.

C13.6.7 HVAC Ductwork. Experience in past earthquakes has shown that HVAC duct systems are rugged and perform well in strong ground shaking. Bracing in accordance with the Sheet Metal and Air Conditioning Contractors National Association ANSI/SMACNA 001 has been effective in limiting damage to duct systems. Typical failures have affected only system function, and major damage or collapse has been uncommon. Therefore, industry standard practices should prove adequate for most installations. Expected earthquake damage is limited to opening of duct joints and tears in ducts. Connection details that are prone to brittle failures, especially hanger rods subject to large amplitude cycles of bending stress, should be avoided.

The amplification factor for ductwork has been increased from 1.0 to 2.5, because even braced duct systems are relatively flexible. The R_p values also have been increased so that the resulting seismic design forces are consistent with those determined previously.

Ductwork systems that carry hazardous materials or must remain operational during and after an earthquake, are assigned a value of I_p =1.5, and require a detailed engineering analysis addressing leak-tightness.

C13.6.8 Piping Systems. In earthquakes, piping systems rarely collapse but often cause nonstructural damage due to leaking. Industry standards and guidelines address a wide variety of piping systems and materials. Construction in accordance with referenced national standards is effective in limiting damage to and avoiding loss of fluid containment in piping systems under earthquake conditions.

ASHRAE's *A Practical Guide to Seismic Restraint,* while not an ANSI standard, is in common use and may be an appropriate reference document for use in the seismic design of piping systems.

The prescriptive conditions provided in the standard under which seismic bracing for piping may be omitted are based on observed performance in past earthquakes.

C13.6.8.1 ASME Pressure Piping Systems. The R_p values tabulated for ASME B31 compliant piping systems reflect the stringent design and quality control requirements as well as the intensified stresses used in ASME design procedures.

C13.6.8.4 Other Piping Systems

Piping not designed in accordance with ASME B31 typically is assigned lower R_p values. Piping component testing suggests that the ductility capacity of carbon steel threaded and grooved joint piping component joints ranges between 1.4 and 3.0. Therefore, these types of connections have been classified as having limited deformability. Grooved couplings and other articulating type of connections may demonstrate free rotational capacity that increases the overall rotational design capacity of the connection. When considered in design, this increase should not exceed 50 percent of the total demonstrated design capacity. The free rotational capacity is the maximum articulating angle where the connection behaves essentially as a pinned joint. The remaining rotational capacity of the connection is where it behaves as a conventional joint whose design force demands are determined by traditional means.

C13.6.9 Boilers and Pressure Vessels

Experience in past earthquakes has shown that boilers and pressure vessels are rugged and perform well in strong ground motion. Construction in accordance with current requirements of the ASME *Boiler and Pressure Vessel Code* (ASME BPVC) has been shown to be effective in limiting damage to and avoiding loss of fluid containment in boilers and pressure vessels under earthquake conditions. It is, therefore, the intent of the standard that nationally recognized codes be used to design boilers and pressure vessels provided that the seismic force and displacement demands are equal to or exceed those outlined in Section 13.3. Where nationally recognized codes do not yet incorporate force and displacement requirements comparable to the requirements of Section 13.3, it is nonetheless the intent to use the design acceptance criteria and construction practices of those codes.

C13.6.10 Elevator and Escalator Design Requirements

The ASME *Safety Code for Elevators and Escalators* (ASME A17.1) has adopted many requirements to improve the seismic response of elevators; however, they do not apply to some regions covered by this chapter. These changes are to extend force requirements for elevators to be consistent with the standard.

C13.6.10.3 Seismic Switches

The purpose of seismic switches as used here is different from that of ASME A17.1, which has incorporated several requirements to improve the seismic response of elevators (such as rope snag point guards, rope retainer guards, and guide rail brackets) and which does not apply to some buildings covered by the standard. Building motions that are expected in areas not covered by the seismic provisions of ASME 17.1 are sufficiently large to impair the operation of elevators. The seismic switch is positioned high in the structure where structural response will be the most severe. The seismic switch trigger level is set to shut down the elevator where structural motions are expected to impair elevator operations.

Elevators in which the seismic switch and counterweight derail device have triggered should not be put back into service without a complete inspection. However, in the case where the loss of use of the elevator creates a life-safety hazard, an attempt to put the elevator back into service may be attempted. Operating the elevator prior to inspection may cause severe damage to the elevator or its components.

The building owner should have detailed written procedures in place defining for the elevator operator/maintenance personnel which elevators in the facility are necessary from a post-earthquake life safety perspective. It is highly recommended that these procedures be in-place, with appropriate personnel training, prior to an event strong enough to trip the seismic switch.

C13.6.10.4 Retainer Plates

The use of retainer plates is a very low cost provision to improve the seismic response of elevators.

C13.6.11 Other Mechanical and Electrical Components. The material properties set forth in Item 2 of this Section are similar to those allowed in ASME BPVC and reflect the high factors of safety necessary for seismic, service, and environmental loads.

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COMMENTARY TO CHAPTER 14, MATERIAL SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS

Because seismic loading is expected to cause nonlinear behavior in structures, seismic design criteria require not only provisions to govern loading, but also provisions to define the required configurations, connections, and detailing to produce material and system behavior consistent with the design assumptions. Thus, while ASCE/SEI 7-05 is primarily a loading standard, compliance with Chapter 14, which covers material specific seismic design and detailing, is required. In general, Chapter 14 adopts material design and detailing standards developed by industry material standards organizations. These materials standards organizations maintain complete commentaries covering their standards and such material is not duplicated here.

The refinements, additions, and recommended changes to the material standards produced by the Provisions Update Committee appear in Part 1 of the 2009 *NEHRP Recommended Seismic Provisions* as exceptions to ASCE/SEI 7-05 along with associated commentary.

C14.0 SCOPE

The scoping statement in this section clarifies that foundation elements are subject to all of the structural design requirements of the standard.

C14.1 STEEL

C14.1.1 Reference Documents. This section lists a series of structural standards published by the American Institute of Steel Construction (AISC), American Iron and Steel Institute (AISI), American Society of Civil Engineers (ASCE/SEI), and Steel Joist Institute (SJI) that are to be applied in the seismic design of steel members and connections in conjunction with the requirements of ASCE/SEI 7. The AISC references are available free of charge in electronic format at www.aisc.org.

C14.1.2 Seismic Design Categories B and C. For the lower Seismic Design Categories B and C, the engineer is allowed a choice in the design of a steel lateral force resisting system. The first option is to design the structure to meet the design and detailing requirements for structures assigned to higher Seismic Design Categories, with the corresponding seismic design parameters (R, Ω_0, and C_d). The second option is to use a lower R factor of 3 (and higher resulting base shear), an Ω_0 of 3, and a C_d value of 3 but without specific seismic design and detailing requirements. The concept of this option is that design for a higher base shear force will result in essentially elastic response that will compensate for the limited ductility of the members and connections, resulting in performance similar to that of more ductile systems.

C14.1.3 Seismic Design Categories D through F. For the higher Seismic Design Categories, the Engineer is not given a choice, but must follow the seismic design provisions of either AISC or AISI using the seismic design parameters specified for the chosen structural system. It is not considered appropriate to design structures without specific design and detailing for seismic response in these high Seismic Design Categories.

C14.1.4 Cold-Formed Steel. This section adopts two standards by direct reference: AISI NAS, *North American Specification for the Design of Cold-Formed Steel Structural Members*, and ASCE/SEI 8, *Specification for the Design of Cold Formed Stainless Steel Structural Members*.

Both of the adopted reference documents have specific limits of applicability. AISI NAS (Section A1.1) applies to the design of structural members that are cold-formed to shape from carbon or low-alloy steel sheet, strip, plate, or bar not more than one-inch in thickness. ASCE/SEI 8 (Section 1.1.1) governs the design of structural members that are cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steels. Both documents focus on load-carrying members in buildings; however, allowances are made for applications in nonbuilding structures, if dynamic effects are considered appropriately.

Within each document, there are requirements related to general provisions for the applicable types of steel; design of elements, members, structural assemblies, connections, and joints; and mandatory testing. In addition, AISI NAS contains a chapter on the design of cold-formed steel structural members and connections undergoing cyclic loading. Both standards contain extensive commentaries for the benefit of the user.

C14.1.4.1 Light-Framed Cold-Formed Steel Construction. This subsection of cold-formed steel relates to light-framed construction, which is defined as a method of construction where the structural assemblies are formed primarily by a system of repetitive wood or cold-formed steel framing members or subassemblies of these members (ASCE/SEI 7-05 Section 11.2). Not only does this subsection repeat the direct adoptions of AISI NAS and ASCE/SEI 8, but it also allows the user to choose from an additional suite of standards that address different aspects of construction, including the following:

1. AISI GP, *Standard for Cold-Formed Steel Framing— General Provisions*, applies to the design, construction, and installation of structural and non-structural cold-formed steel framing members where the specified minimum base metal thickness is between 18 mils and 118 mils (Section A1).
2. AISI WSD, *Standard for Cold-Formed Steel Framing – Wall Stud Design,* applies to the design and installation of cold-formed steel studs for both structural and non-structural walls in buildings (Section A1).
3. AISI Lateral, *Standard for Cold-Formed Steel Framing – Lateral Design,* contains design requirements for shear walls, diagonal strap bracing (as part of a structural wall), and diaphragms (Section A1).

The requirements of AISI GP apply to all light-framed cold-formed steel and, consequently, the standard is adopted by direct reference in both AISI WSD and AISI Lateral. In addition, all of these documents include commentaries to aid the user in the correct application of their requirements.

C14.1.5 Prescriptive Framing. This section adopts AISI PM, *Standard for Cold-Formed Steel Framing – Prescriptive Method for One and Two Family Dwellings*, which applies to the construction of detached one-and two-family dwellings, townhouses, and other attached single-family dwellings not more than two stories in height using repetitive in-line framing practices (Section A1). This document adopts AISI GP by direct reference and includes a commentary to aid the user in the correct application of its requirements.

C14.1.6 Steel Deck Diaphragms. Design of steel deck diaphragms is to be based upon recognized national standards or a specific testing program directed by a person experienced in testing procedures and steel deck. All fastener design values (welds, screws, power actuated fasteners, button punches) for attaching steel deck sheet to steel deck sheet or for attaching the steel deck to the building framing members must be per recognized national design standards or specific steel deck testing programs. All steel deck diaphragm and fastener design properties must be approved for use by the authorities in whose jurisdiction the construction project occurs. Steel deck diaphragm in-plane design forces (seismic, wind, or gravity) must be determined per ASCE/SEI 7-05 Section 12.10.1. Steel deck manufacturer test reports prepared in accordance with this provision can be used where adopted and approved by the authority having jurisdiction for the building project. The diaphragm design manual produced by the Steel Deck Institute (2004) is also a potential reference for design values.

Steel deck is assumed to have a corrugated profile consisting of alternating up and down flutes that are manufactured in various widths and heights. Use of flat sheet metal as the overall floor or roof diaphragm is permissible where designed by engineering principles, but is beyond the scope of this section. Flat or bent sheet metal may be used as closure pieces for small gaps or penetrations or for shear transfer over short distances in the steel deck diaphragm where diaphragm design forces are considered.

Steel deck diaphragm analysis must include design of chord members at the perimeter of the diaphragm and around interior openings in the diaphragm. Chord members may be steel beams attached to the underside of the steel deck designed for a combination of axial loads and bending moments due to acting gravity and lateral loads.

Where diaphragm design loads exceed the bare steel deck diaphragm design capacity, then either horizontal steel trusses or a structurally designed concrete topping slab placed over the steel deck must be provided to distribute lateral forces. Where horizontal steel trusses are used, the steel deck must be designed to transfer diaphragm forces to the steel trusses. Where a structural concrete topping over the steel deck is used as the diaphragm, the diaphragm chord members at the perimeter of the diaphragm and edges of interior openings must be either: (a) designed flexural reinforcing steel placed in the structural concrete topping or (b) steel beams located under the steel deck with connectors (that provide a positive connection) as required to transfer design shear forces between the concrete topping and steel beams.

C14.1.7 Steel Cables. These provisions reference ASCE/SEI 19-96, *Structural Applications of Steel Cables for Buildings*, for the determination of the design strength of steel cables. ASCE/SEI 19 uses service level load combinations with a safety factor relative to the cable design strength. The service level load combinations specified in ASCE/SEI 19 are adjusted in two ways. First, the prestress loading is multiplied by a factor of 1.1 to account for any over prestressing that may occur in the field. Second, the safety factor for load combinations including seismic effects is reduced from 2.0 to 1.5 to account for the dynamic nature of seismic loading and the ductility of the system. While T3 and T4 in ASCE/SEI 19 may be calculated using either wind or seismic loads, the modifications of this section apply only to load combinations including seismic loadings.

C14.1.8 Additional Detailing Requirements for Steel Piles in Seismic Design Categories D through F. Steel piles used in higher Seismic Design Categories are expected to yield just under the pile cap or foundation due to combined bending and axial load. Design and detailing requirements of AISC 341 for H-piles are intended to produce stable plastic hinge formation in the piles. Since piles can be subjected to tension due to overturning moment, mechanical means to transfer such tension must be designed for the required tension force, but not less than 10 percent of the pile compression capacity.

C14.2 CONCRETE

The section adopts ACI 318-05, *Building Code Requirements for Structural Concrete* (ACI 318), by reference for structural concrete design and construction. In addition, modifications to ACI 318 are made to coordinate the provisions of that material design standard with the provisions of ASCE/SEI 7.

C14.2.2.1 ACI 318 Section 7.10. The reinforcement details for ties in compression members prescribed in ACI 318 Section 7.10.5 are appropriate for SDC A and B structures. This modification prescribes additional details for ties around anchor bolts of structures assigned to SDC C, D, E, or F.

C14.2.2.2 ACI 318 Section 10.5. This provision affects ordinary moment frames. It is intended to improve continuity, and thereby lateral force resistance and structural integrity, compared to that of frames designed to the provisions of Chapters1 through 18 of ACI 318 only. The provision does not apply to slab-column moment frames.

C14.2.2.3 ACI 318 Section 11.11. This requirement is intended to provide additional toughness to resist shear for columns of frames in SDC B. Otherwise the proportions of those columns make them more susceptible to shear failure under earthquake loading.

C14.2.2.4 Definitions. The first four definitions relate the wall types of ASCE/SEI 7-05 with detailing requirements of ACI 318 and distinguish between ordinary reinforced concrete structural walls and ordinary precast structural walls. These definitions are essential to the proper interpretation of the R and C_d factors for each wall type specified in Table 12.2-1.

A wall pier is recognized as a separate category of structural element in this document but not in ACI 318.

C14.2.2.5 Scope. ACI 318 uses the terminology of low, moderate, and high seismic risk for structures assigned to SDC A and B, SDC C, and SDC D through F, respectively. The modifications of this provision show how the ACI 318 provisions should be interpreted for consistency with the ASCE/SEI 7-05 provisions.

C14.2.2.6 Reinforcement in Members Resisting Earthquake-Induced Forces. ACI 318 does not allow the use of prestressing tendons in special and intermediate moment frames. This provision and ASCE/SEI 7-05 Sections 14.2.2.7 and 14.2.2.8 impose conditions that have been demonstrated to permit the safe use of such tendons.

These provisions are intended to apply to frames containing unbonded tendons only. The average prestress in plastic hinge regions is restricted to limit the strain in the prestressing steel under the design displacement to not greater than 1 percent. The strain in the prestressing steel at the design displacement should be calculated considering the anticipated inelastic mechanism of the structure.

C14.2.2.7 Anchorages for Unbonded Post-tensioning Tendons. Fatigue testing for 50 cycles of loading between 40 and 80 percent of the specified tensile strength of the prestressing strand has been an industry practice of long standing (ACI 423.6, *Specification for Unbonded Single-Strand Tendons*). The 80 percent limit is increased to 85 percent for seismic applications in order to correspond to a 1 percent limit, and therefore the effective start of yielding, in the prestressing steel. Testing over this range of stress conservatively simulates the effect of a severe earthquake on structures prestressed in accordance with the requirements of ASCE/SEI 7-05 Sections 14.2.2.6 and 14.2.2.8.

C14.2.2.8 Flexural Members of Special Moment Frames. The restrictions on the flexural strength provided by the tendons are based on the results of analytical and experimental studies (Ishizuka and Hawkins, 1987; Park and Thompson, 1977). Although satisfactory seismic performance can be obtained with greater amounts of prestressing steel, this restriction is needed to allow the use of the same response modification and deflection amplification factors as those specified for special moment frames without prestressing steel.

C14.2.2.9 Wall Piers and Wall Segments. Wall piers are typically segments between openings in walls that are thin in the direction normal to the face of the wall. In current practice these elements are often not regarded as columns or as part of the special structural walls. If not properly reinforced these elements are vulnerable to shear failure, and that failure prevents the wall from developing the assumed flexural hinging. ACI 318 Section 21.7.10 is written specifically to preclude such pre-emptive shear failure. The required shear strength in ACI 318 Section 21.4.5.1 is based on the probable shear strength, V_e , under the probable moment, M_{pr}. Wall segments with a horizontal length-to-thickness ratio less than 2.5 and a clear height-to-length ratio of at least 2 are required to be designed as columns in compliance with ACI 318 Section 21.4 if they are used as part of the lateral-force-resisting system even though the shortest cross-sectional dimension may be less than 12 inches in violation of Section 21.4.1.1. Such wall segments may be designed to comply with ACI 318 Section 21.11 if they are not used as part of the lateral-force-resisting system. Wall segments with a horizontal length-to-thickness ratio larger than or equal to 2.5, which do not meet the definition of wall piers (ASCE/SEI 7-05 Section 14.2.2.4), must be designed as special structural walls or as portions of special structural walls in full compliance with ACI 318 Section 21.7.

C14.2.2.12 Members Not Designated as Part of the Lateral-Force-Resisting System. ACI 318 Section 21.4.3.2 permits lap splices only within the center half of the column. Section 21.11.2 applies where the magnitude of the moments induced in the column by the design displacement are explicitly checked. Section 21.11.3 applies where the effects of the design displacement are not explicitly checked. Section 21.11.2.2, if not modified, would permit lap splices to be placed at any location over the height of the column if the column is expected to yield. If, however, the column is not expected to yield the wording effectively requires the splice to be located near mid-height. This is not rational and the modification results in a more rational provision.

C14.2.2.13 Columns Supporting Reactions from Discontinuous Stiff Members. Discontinuous shear walls and other stiff members can impose large axial forces on supporting columns. The specified transverse reinforcement is to improve column toughness under anticipated seismic demands.

C14.2.2.14 Intermediate Precast Structural Walls. ACI 318 Section 21.13 imposes requirements on precast walls for moderate seismic risk applications. The intent is to produce ductile behavior by yielding of the steel elements or reinforcement between panels or between panels and foundations. The 2003 IBC restricted yielding to steel reinforcement because of concern that steel elements in the body of a connection could fracture due to strain demands.

Several steel element connections have been tested under simulated seismic loading and the adequacy of their load-deformation characteristics and strain capacity of yield has been demonstrated (Schultz and Magana, 1996). One such connection was used in the five-story building test that was part of the PRESSS Phase 3 research. The connection was used to provide damping and energy dissipation, and demonstrated a very large strain capacity (Nakaki et al., 2001). Since then several other steel element connections have been developed that can achieve similar results (Banks and Stanton, 2005; Nakaki et al., 2005). In view of these results it is appropriate to allow yielding in steel elements that have been shown experimentally to have adequate strain capacity to maintain at least 80 percent of their yield force of through the full design displacement of the structure. This provision requires the designer to determine the deformation in the connection corresponding to the earthquake design displacement, and then to check for experimental data that the connection type used can accommodate that deformation without significant strength degradation.

The wall pier requirements in the modified ACI 318 Section 21.13.5 are less stringent than those for wall piers for special structural walls as specified in the modified Section 21.7.10. Where intermediate precast structural walls are used in SDCs D, E and F, wall piers should satisfy the requirements of ASCE/SEI 7-05 Section 14.2.2.9 rather than 14.2.2.14.

C14.2.2.15 Detailed Plain Concrete Shear Walls. Design requirements for plain masonry walls have existed for many years, and the competing type of concrete construction is the plain concrete wall. To allow the use of such walls as the lateral-force-resisting system in SDC A and B, this provision requires such walls to contain at least the minimal reinforcement specified in ACI 318 Section 22.6.7.2.

C14.2.2.16 Plain Concrete in Structures Assigned to Seismic Design Category C, D, E, or F. Modifications are made to ACI 318 Section 22.10 that restrict markedly the use of ordinary and detailed structural plain concrete walls in SDC C, D, E, and F.

C14.2.2.17 General Requirements for Anchoring to Concrete. ACI 318 uses the terminology of regions of moderate or high seismic risk and structures assigned to intermediate or high seismic performance or design categories. In this modification, the only changes to ACI 318 in Sections D3.3.3 through D3.3.4 are the replacement of that terminology with the SDC terminology.

There are two changes to the provisions in ACI 318 Section D3.3.5. The first is the use of the SDC terminology, and the second is the addition of the last phrase of the provision referring to the minimum design strength of the anchors. The last phrase requires an anchor strength that is at least the maximum likely Ω_0 value (2.5) times the design force calculated as being transmitted to the attachment by the lateral-force-resisting system.

C14.2.2.18 Strength Requirements for Anchors. ACI 318 requires laboratory testing to establish the strength of anchor bolts greater than 2 inches in diameter or exceeding 25 inches in tensile embedment depth. This modification makes the ACI 318 equation giving the basic concrete breakout strength of a single anchor in tension in cracked concrete applicable irrespective of the anchor bolt diameter and tensile embedment depth.

Korean Power Engineering (KPE) has made tension tests on anchors with diameters up to 4.25 inches and embedment depths up to 45 inches and found that the diameter and embedment depth limits of ACI 318 Section D4.2.2 for the design procedure for anchors in tension (Section D5.2) can be eliminated. KPE has also made shear tests on anchors with diameters up to 3.0

inches and embedment depths as large as 30 inches and found no effect of the embedment depth on shear strength. However, the diameter tests showed that the basic shear breakout strength equation (ACI 318 Section D-24) needed some modification for the complete elimination of the 2 inch limit to be fully appropriate. Analytical work performed at the University of Stuttgart supports the need for some modification to the ACI 318 Equation D-24. Changes consistent with the Korean and Stuttgart findings have already been made to the FIB Design Guide for anchors and a change proposal consistent with those changes has been submitted to ACI 318 for consideration.

C14.2.3.1.2 Reinforcement for Uncased Concrete Piles (SDC C): The transverse reinforcing requirements in the potential plastic hinge zone of uncased concrete piles in Seismic Design Category C is a selective composite of two ACI 318 requirements. In the potential plastic hinge region of an intermediate moment-resisting concrete frame column, the transverse reinforcement spacing is restricted to the least of: (a) 8 times the diameter of the smallest longitudinal bar, (b) 24 times the diameter of the tie bar, (c) one-half the smallest cross-sectional dimension of the column, and (d) 12 inches. Outside of the potential plastic hinge region of a special moment-resisting frame column, the transverse reinforcement spacing is restricted to the smaller of: 6 times the diameter of the longitudinal column bars and 6 inches.

C14.2.3.1.5 Reinforcement for Precast Nonprestressed Concrete Piles (SDC C): Transverse reinforcement requirements inside and outside of the plastic hinge zone of precast nonprestressed piles are clarified. The transverse reinforcement requirement in the potential plastic hinge zone is a composite of two ACI 318 requirements (see Section C14.2.3.1.2). Outside of the potential plastic hinge region the eight longitudinal-bar-diameter spacing is doubled. The maximum 8-in. tie spacing comes from current building code provisions for precast concrete piles.

C14.2.3.1.6 Reinforcement for Precast Prestressed Piles (SDC C): The transverse and longitudinal reinforcing requirements given in ACI 318 Chapter 21 were never intended for slender precast prestressed concrete elements and will result in unbuildable piles. The requirements are based on the 1993 *Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling* by the PCI Committee on Prestressed Concrete Piling.

ASCE/SEI 7-05 Equation 14.2-1, originally from ACI 318, has always been intended to be a lower-bound spiral reinforcement ratio for larger diameter columns. It is independent of the member section properties and therefore can be applied to large or small diameter piles. For cast-in-place concrete piles and precast prestressed concrete piles, the resulting spiral reinforcing ratios from this formula are considered to be sufficient to provide moderate ductility capacities.

Full confinement per Equation 14.2-1 is required for the upper 20 feet of the pile length where curvatures are large. The amount is relaxed by 50 percent outside of that length in view of lower curvatures and in consideration of confinement provided by the soil.

C14.2.3.2.5 Reinforcement for Precast Concrete Piles (SDC D through F): The transverse reinforcement requirements for precast nonprestressed concrete piles are taken from current building code requirements and are intended to provide ductility in the potential plastic hinge zones.

C14.2.3.2.6 Reinforcement for Precast-Prestressed Piles (SDC D through F): The last paragraph provides minimum transverse reinforcement outside of the zone of prescribed ductile detailing.

C14.3 COMPOSITE STEEL AND CONCRETE STRUCTURES

This section provides guidance on the design of composite and hybrid steel-concrete structures. Composite structures are defined as those incorporating structural elements made of steel and concrete portions connected integrally throughout the structural element by mechanical connectors, bond, or both. Hybrid structures are defined as consisting of steel and concrete structural elements connected together at discrete points. Composite and hybrid structural systems mimic many of the existing steel (moment and braced frame) and concrete (moment frame and wall) configurations, but are given their own design coefficients and factors in Table 12.2-1. Their design is based on the same ductility and energy dissipation concepts used in conventional steel and reinforced concrete structures, but requires special attention to the interaction of the two materials as it affects the stiffness, strength, and inelastic behavior of the members, connections, and systems.

C14.3.1 Reference Documents. Seismic design for composite structures assigned to Seismic Design Category D, E, or F is governed primarily by *Part II: Composite Structural Steel and Reinforced Concrete Buildings* of ANSI/AISC 341. Part II of ANSI/AISC 341 is less prescriptive than Part I and provides flexibility for designers to utilize analytical tools and results of research in their practice. Composite structures assigned to Seismic Design Category A, B, or C may be designed according to principles outlined in ANSI/AISC 360 and ACI 318. ACI 318 and ANSI/AISC 360 provide little guidance on connection design; therefore, designers are encouraged to review ANSI/AISC 341 Part II for guidance on the design of joint areas. Differences between older AISC and ACI provisions for cross-sectional strength for composite columns have been minimized by changes in the latest ANSI/AISC 360. However, there is not uniform agreement between the provisions in

ACI 318 and ANSI/AISC 360 regarding detailing, limits on material strengths, stability, and shear design for composite columns. The composite design provisions in ANSI/AISC 360 are considered to be current.

C14.3.2 Metal-Cased Concrete Piles. Design of metal-cased concrete piles, which are analogous to circular concrete filled tubes, is governed by ASCE/SEI 7-05 Sections 14.2.3.1.3 and 14.2.3.2.4. The intent of these provisions is to require metal-cased concrete piles to have confinement and protection against long-term deterioration comparable to that for uncased concrete piles.

C14.4 MASONRY

Seismic design for masonry structures is governed primarily by two documents produced by the Masonry Standards Joint Committee (MSJC): ACI 530-05/ASCE/SEI 5-05/TMS 402-5, *Building Code Requirements for Masonry Structures* (the MSJC Code), and ACI 530.1-05/ASCE/SEI 6-05/TMS 602-05, *Specification for Masonry Structures* (the MSJC Specification).

C14.4.2 *R* Factors. Where intermediate and special reinforced masonry shear walls are designed using the allowable-stress provisions of the MSJC Code, these additional requirements are intended to produce a level of inelastic flexural deformation capacity consistent with that of intermediate and special reinforced masonry shear walls designed using the strength-design provisions of the MSJC Code. The additional requirements are discussed in ASCE/SEI 7-05 Section C14.4.6.

C14.4.3 Classification of Shear Walls. Section 1.14 of the 2005 MSJC Code can be interpreted as permitting, in SDCs A and B, masonry walls that need not be considered part of the lateral-force-resisting system and that do not need to be isolated. ASCE/SEI 7-05 Section 14.4.3 is intended to preclude that interpretation.

C14.4.5.1 Separation Joints. This section is intended to address force transfer across interfaces between masonry and other materials, but it is redundant. Article 3.2B of the MSJC Specification requires that the interface between concrete and masonry be cleaned and acceptable for laying of units. Further, Section 1.9.4.2.4 of the 2005 MSJC Code addresses the design and transfer of shear at interfaces, and Section 1.7.5.2 requires that a load path and force transfer between a foundation and the masonry above be maintained.

C14.4.5.2 Flanged Shear Walls. Section 1.9.4.2.3 of the MSJC Code contains the compression requirement (lesser of 6 times the flange thickness or the actual flange). The principal effect of the tension provision in ASCE/SEI 7-05 Section 14.4.5.2 is to establish the amount of tensile reinforcement used in calculating flexural capacity and maximum permitted reinforcement, but this provision is not well established technically. Research in masonry, and analogous design provisions for concrete (ACI 318 Section 21.7.5.2), suggest that effective flange widths in tension are more logically related to the total wall height rather than the floor-to-floor height. The 2005 MSJC Code and ASCE/SEI 7-05 are working together to resolve this issue and add appropriate requirements to TMS 402.

C14.4.6 Modifications to Chapter 2 of ACI 530/ASCE/SEI 5/TMS 402. Chapter 2 of the MSJC Code deals with allowable-stress design.

C14.4.6.1 Stress Increase. The MSJC Code permits allowable stresses to be increased by one-third for allowable-stress loading conditions that include wind or earthquake, provided that the legally adopted building code so permits. While the alternate allowable-stress loading combinations of the 2006 IBC do so permit, the allowable-stress loading combinations of ASCE/SEI 7-05 do not.

C14.4.6.2 Reinforcement Requirements and Details.

C14.4.6.2.1 Reinforcing Bar Size Limitations. The intent of this requirement is to prevent splitting of masonry due to the presence of reinforcement. A similar requirement is appears in Chapter 3 (Strength Design) of the 2005 MSJC Code. The MSJC is working to move that requirement to Chapter 1 (General Requirements) so that it would apply to all masonry construction.

C14.4.6.2.2 Splices. In general, the first portion of this section, which prohibits splices in plastic hinge zones, is intended to produce adequate inelastic deformation capacity in those regions. In general, the presence of splices in plastic hinge zones reduces inelastic deformation capacity because the area of steel is doubled at the splice, reducing the extent of yielding. However, there is some controversy concerning the technical validity and necessity for this requirement for masonry walls. Similar requirements apply to plastic hinge zones of reinforced concrete frames, they do not apply to plastic hinge zones of reinforced concrete walls. Also, this requirement does not distinguish between shear-critical and flexurally dominated shear walls. The MSJC is continuing to discuss related requirements for flexurally dominated, highly ductile shear walls.

The remaining portions of this section (requirements for splices) are intended to provide adequate capacity of welded splices and mechanical connections. The MSJC is developing similar provisions.

C14.4.6.2.3 Maximum Area of Flexural Tensile Reinforcement. The intent of this section is to produce adequate inelastic flexural deformation capacity in flexurally dominated masonry shear walls by placing an upper limit on flexural reinforcement, so that behavior is dominated by yielding of reinforcement rather than by crushing of the compression toe. Similar provisions appear in Chapter 3 (Strength Design) of the MSJC Code and are being developed for Chapter 2 (Allowable-Stress Design).

C14.4.7 Modifications to Chapter 3 of ACI 530/ASCE/SEI 5/TMS 402.

C14.4.7.2 Splices in Reinforcement. See Section C14.4.6.2.2.

C14.4.7.3 Coupling Beams. The intent of this requirement is to produce adequate inelastic flexural deformation capacity in coupling beams. The section is somewhat redundant with Section 3.1.3 of the MSJC Code, which requires capacity design of masonry elements for shear.

C14.4.7.4 Deep Flexural Members. The intent of this requirement is to require that the design of deep flexural members correctly addresses the presence of distributed flexural reinforcement in capacity design for shear, and that crack widths are adequately controlled.

C14.4.7.5 Shear Keys. The intent of this requirement is to increase resistance to sliding shear at the foundation level of flexurally dominated masonry shear walls. The original proposal was based on laboratory research (Leiva et al., 1990) involving isolated shear walls. In subsequent research (Seible et al., 1993), flanged walls without shear keys did not show sliding.

C14.4.7.6 Anchoring to Masonry. The intent of this requirement is to guard against brittle failure of masonry anchorages that are part of the seismic force-resisting system.

C14.4.7.7 Anchor Bolts. ASCE/SEI 7-05 Sections 14.4.7.7 and 14.4.7.8 augment the current anchor bolt provisions of MSJC Code Chapter 3 (Strength Design) to address pryout and to include an appropriate ϕ factor.

C14.4.8 Modifications to Chapter 6 of ACI 530/ASCE/SEI 5/TMS 402. There is an apparent difference in the treatment of corrugated sheet metal anchors in different chapters of the MSJC Code. Chapter 6 of that document, dealing with masonry veneer, permits corrugated sheet-metal anchors. Chapters 2 and 3 of that document do not permit multi-wythe, noncomposite masonry (functionally identical to veneer) to be bonded by corrugated sheet-metal anchors.

C14.4.9 Modifications to ACI 530.1/ASCE/SEI 6/TMS 602.

C14.4.9.1 Construction Procedures. This requirement was introduced originally as a result of the TCCMaR program as a way to address volume loss as a result of plastic shrinkage of grout. The original provision required the use of a particular admixture (Sika's Grout Aid®) in the grout. The MSJC Specification requires both consolidation and reconsolidation of masonry grout, which in combination with today's masonry construction materials can minimize grout shrinkage without the requirement of a proprietary grout admixture available from a single source.

C14.5 WOOD

C14.5.1 Reference Documents. Two national consensus standards are adopted for seismic design of engineered wood structures: the *National Design Specification* (NDS), and the *Special Design Provisions for Wind and Seismic* (SDPWS) Supplement to the NDS. Both of these standards, published by the American Forest and Paper Association (AF&PA), are presented in dual allowable stress design (ASD) and load and resistance factor design (LRFD) formats. Both standards reference a number of secondary standards for related items such as wood materials and fasteners. SDPWS addresses general principles and specific detailing requirements for shear wall and diaphragm design and provides tabulated nominal unit shear capacities for shear wall and diaphragm sheathing and fastening. The balance of member and connection design is to be in accordance with the NDS. A commentary to the NDS is published by AF&PA (2005b); commentary to the SDPWS is included in the SDPWS publication (AF&PA, 2005c).

C14.5.2 Framing. This section provides specific guidance on two general topics related to detailing. First, vertical loads on columns and posts must be transferred in and out by end bearing only or by connectors only; mixing the capacity of end bearing and connectors is prohibited due to a potential lack of deformation compatibility. Second, load path continuity for top plates, which often function as collectors, is addressed.

C14.5.3.1 ASCE/SEI 7-05 Modification to SDPWS Section 4.3.3.2, Summing Shear Capacities. This amendment to the SDPWS does not provide additional clarity; therefore, it is expected to be deleted ASCE/SEI 7-10.

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COMMENTARY TO CHAPTER 15, SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES

C15.1.1 Nonbuilding Structures. Building codes traditionally have been perceived as minimum standards for the design of nonbuilding structures, and building code compliance of these structures is required by building officials in many jurisdictions. However, requirements in the industry reference documents are often at odds with building code requirements. In some cases, the industry documents need to be altered while in other cases the building codes need to be modified. Registered design professionals are not always aware of the numerous accepted documents within an industry and may not know whether the accepted documents are adequate. The intent of Chapter 15 of the standard is to bridge the gap between building codes and existing industry reference documents.

Differences between the ASCE/SEI 7-05 design approaches for buildings and industry document requirements for steel multi-legged water towers (Figure C15.1-1) are representative of this inconsistency. Historically, such towers have performed well when properly designed in accordance with American Water Works Association (AWWA) standards and industry practices. Those standards and practices differ from the ASCE/SEI 7-05 treatment of buildings in that tension-only rods are allowed, upset rods are preloaded at the time of installation, and connection forces are not amplified.

Figure C15.1-1 Steel multi-legged water tower.

Chapter 15 also provides an appropriate link so that the industry reference documents can be used with the seismic ground motions established in the standard. It should be noted that some nonbuilding structures are very similar to buildings and can be designed employing sections of the standard directly, whereas other nonbuilding structures require special analysis unique to the particular type of nonbuilding structure.

Note that building structures, vehicular bridges, electrical transmission towers, hydraulic structures (e.g., dams), buried utility lines and their appurtenances, and nuclear reactors are excluded from the scope of the nonbuilding structure requirements. The excluded structures are covered by other well established design criteria (e.g., electrical transmission towers and vehicular bridges), are not under the jurisdiction of local building officials (e.g., nuclear reactors, and dams), or require technical considerations beyond the scope of the standard (e.g., buried utility lines and their appurtenances).

C15.1.2 Design. Nonbuilding structures and building structures have much in common with respect to design intent and expected performance, but there are also important differences. Chapter 15 relies on other portions of the standard where possible and provides special notes where necessary.

There are two types of nonbuilding structures: those with structural systems similar to buildings, and those with structural systems not similar to buildings. Specific requirements for these two cases appear in Sections 15.5 and 15.6.

C15.1.3 Structural Analysis Procedure Selection. Nonbuilding structures that are similar to buildings are subject to the same analysis procedure limitations as building structures. Nonbuilding structures that are not similar to buildings are subject to those limitations and are subject to procedure limitations prescribed in applicable specific reference documents.

For many nonbuilding structures supporting flexible system components, such as pipe racks (Figure C15.1-2), the supported piping and platforms generally are not regarded as rigid enough to redistribute seismic forces to the supporting frames.

Figure C15.1-2 Steel pipe rack.

For nonbuilding structures supporting rigid system components, such as steam turbine generators (STGs) and heat recovery steam generators (HRSGs) (Figure C15.1-3), the supported equipment, ductwork, and other components (depending on how they are attached to the structure) may be rigid enough to redistribute seismic forces to the supporting frames. Torsional effects may need to be considered in such situations.

Section 12.6 presents seismic analysis procedures for building structures based on the Seismic Design Category; the fundamental period, T; and the presence of certain horizontal or vertical irregularities in the structural system. Where the fundamental period is greater than or equal to $3.5T_s$ (where $T_s = S_{D1}/S_{DS}$), the use of the equivalent lateral force procedure is not permitted in Seismic Design Categories D, E, and F. This requirement is based on the fact that, unlike the dominance of the first mode response in case of buildings with lower first mode period, higher vibration modes do contribute more significantly in situations when the first mode period is larger than $3.5Ts$. For buildings that exhibit classic flexural deformation patterns (such as slender shear wall or braced frame systems), the second mode frequency is at least 3.5 times the first mode frequency, so where the fundamental period exceeds $3.5Ts$, the higher modes will have larger contributions to the total response as they occur near the peak of the design response spectrum

It follows that dynamic analysis (modal response spectrum analysis, or response-history analysis) is required for building-like nonbuilding structures if the first mode period is larger than $3.5Ts$ and that the equivalent lateral force analysis is sufficient for nonbuilding structures that respond as single-degree-of-freedom systems such as single-pedestal elevated water tanks.

The recommendations for nonbuilding structures provided below are intended to supplement the designer's judgment and experience. The designer is given considerable latitude in selecting a suitable analysis method for nonbuilding structures.

Figure C15.1-3 Heat recovery steam generators.

Building-like Nonbuilding Structures. Table 12.6-1 is used in selecting analysis methods for building-like nonbuilding structures, but, as illustrated in the following three conditions, the relevance of key behavior must be considered carefully:

1. Irregularities: Table 12.6-1 requires dynamic analysis for Seismic Design Category D, E, and F structures having certain horizontal or vertical irregularities. Some of these building irregularities (defined in Section 12.3.2) are relevant to nonbuilding structures. The weak-and soft-story vertical irregularities (Types 1a, 1b, 5a, and 5b of Table 12.3-2) are pertinent to the behavior of building-like nonbuilding structures. Other vertical and horizontal irregularities may or may not be relevant as described below.

    a. Horizontal irregularities: Horizontal irregularities of Type 1a and 1b affect the choice of analysis method, but these irregularities apply only where diaphragms are rigid or semirigid and some building-like nonbuilding structures have either no diaphragms or flexible diaphragms.

    b. Vertical irregularities: Vertical irregularity Type 2 is relevant where the various levels actually support significant loads. Where a building-like nonbuilding structure supports significant mass at a single level, while other levels support small masses associated with stair landings, access platforms, and so forth, dynamic response will be dominated by the first mode, so the equivalent lateral force procedure may be applied. Vertical irregularity Type 3 addresses large differences in the horizontal dimension of the seismic force-resisting system in adjacent stories, since the resulting stiffness distribution can produce a fundamental mode shape unlike that assumed in the development of the equivalent lateral force procedure. Since the concern relates to stiffness distribution, it is the horizontal dimension of the seismic force-resisting system, not of the overall structure, that is important.

2. Arrangement of supported masses: Even where a nonbuilding structure has building-like appearance, it may not behave like a building, depending on how masses are attached. For example, the response of nonbuilding structures with suspended vessels and boilers cannot be determined reliably using the equivalent lateral force procedure because of the pendulum modes associated with the significant mass of the suspended components. The resulting pendulum modes, while potentially reducing story shears and base shear, may require large clearances to allow pendulum motion of the supported components and may produce excessive demands on attached piping. Dynamic analysis should be performed in such cases, with consideration for appropriate impact forces in the absence of adequate clearances.

3. Relative rigidity of beams: Even where a classic building model may seem appropriate, the equivalent lateral force procedure may underpredict the total response if the beams are flexible relative to the columns (of moment frames) or the braces (of braced frames). This is because higher modes associated with beam flexure may contribute more significantly to the total response (even if the first mode response is at a period less than $3.5T_S$). This situation of flexible beams can be especially pronounced for nonbuilding structures since the "normal" floors common to buildings may be absent. Therefore, the dynamic analysis procedures are recommended for building-like nonbuilding structures with flexible beams.

Nonbuilding Structures Not Similar to Buildings. The (static) equivalent lateral force procedure is based on classic building dynamic behavior, which is an inappropriate characterization for many nonbuilding structures not similar to buildings. As discussed below, several issues should be considered for selecting either an appropriate method of dynamic analysis or a suitable distribution of lateral forces for static analysis.

1. Structural geometry: The dynamic response of nonbuilding structures with a fixed base and a relatively uniform distribution of mass and stiffness, such as bottom-supported vertical vessels, stacks, and chimneys, can be represented adequately by a cantilever (shear building) model. For these structures the equivalent lateral force procedure provided in the standard is suitable. This procedure treats the dynamic response as being dominated by the first mode. In such cases, it is necessary to identify the first mode shape (using, for instance, the Rayleigh-Ritz method or other classical methods from the literature) for distribution of the dynamic forces. For some structures, such as tanks with low height-to-diameter ratios storing granular solids, it is conservative to assume a uniform distribution of forces. Dynamic analysis is recommended for structures that have neither a uniform distribution of mass and stiffness nor an easily determined first mode shape.

2. Number of lateral supports: Cantilever models are obviously unsuitable for structures with multiple supports. Figure C15.1-4 shows a nonbuilding braced frame structure that provides non-uniform horizontal support to a piece of equipment. In such cases, the analysis should include coupled model effects. For such structures an application of the equivalent lateral force method could be used depending on the number and locations of the supports. For example, most beam-type configurations lend themselves to application of the equivalent lateral force method.

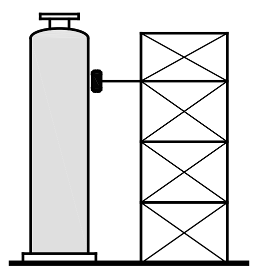

Figure C15.1-4 Multiple lateral supports.

3. Method of supporting dead weight: Certain nonbuilding structures (such as power boilers) are supported from the top. They may be idealized as pendulums with uniform mass distribution. In contrast, a suspended platform may be idealized as a classic pendulum with concentrated mass. In either case, these types of nonbuilding structures can be analyzed adequately using the equivalent lateral force method by calculating the appropriate frequency and mode shape. Figure C15.1-5 shows a nonbuilding structure containing lug supported equipment with W_P greater than $0.25(W_S + W_P)$. In such cases, the analysis should include a coupled system with the mass of the equipment and the local flexibility of the supports considered in the model. Where the support is located near the nonbuilding structure's vertical location of the center of mass, a dynamic analysis is recommended.

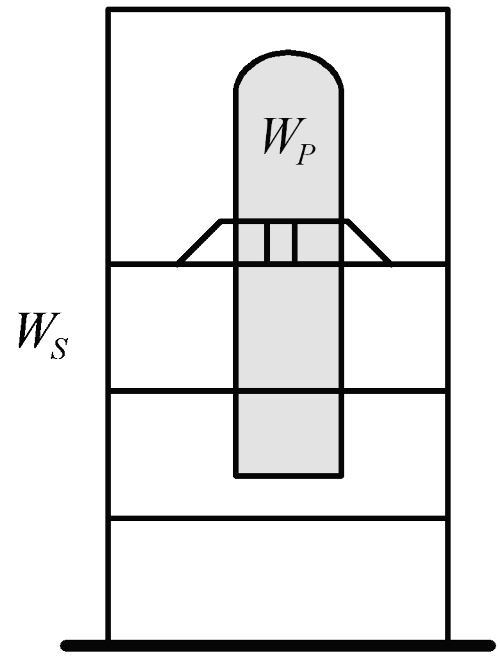

Figure C15.1-5 Unusual support of dead weight.

4. Mass irregularities: Just as in the case of building-like nonbuilding structures, the presence of significantly uneven mass distribution can render the structures unsuitable for application of the equivalent lateral force method. The dynamic analysis methods are recommended in such situations. Figure C15.1-6 illustrates two such situations. In part (a), a mass irregularity exists if W_1 is greater than $1.5W_2$ or less than $0.67W_2$. In part (b), a mass irregularity exists if W_3 is greater than either $1.5W_2$ or $1.5W_4$.

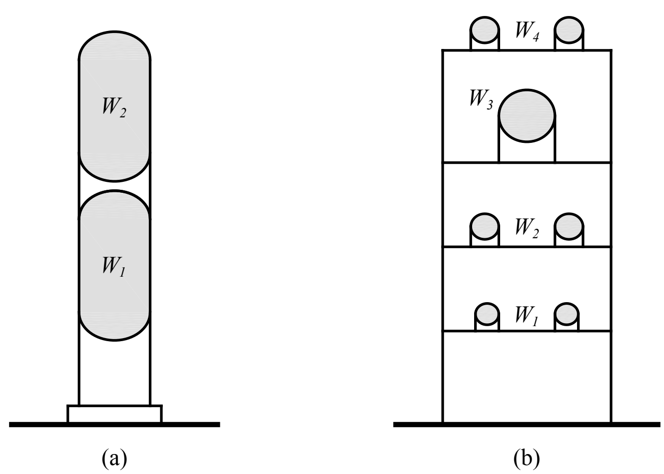

Figure C15.1-6 Mass irregularities.

5. Torsional irregularities: Structures in which the fundamental mode of response is torsional or in which modes with significant mass participation exhibit a prominent torsional component may also have inertial force distributions that are significantly different from that predicted by the equivalent lateral force method. In such cases dynamic analyses should be considered. Figure C15.1-7 illustrates one such case where a vertical vessel is attached to a secondary vessel with W_2 greater than about $0.25(W_1 + W_2)$.

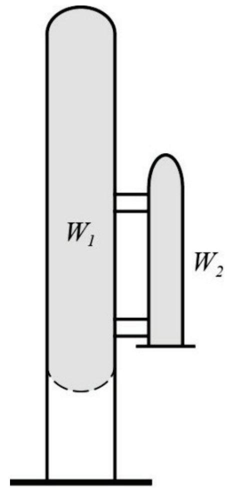

Figure C15.1-7 Torsional irregularity.

6. Stiffness/strength irregularities: Just as for building-like nonbuilding structures, abrupt changes in the distribution of stiffness or strength in a nonbuilding structure not similar to buildings can result in substantially different inertial forces that differ substantially from those indicated by the equivalent lateral force method. Figure C15.1-8 represents one such case. For structures having such configurations, consideration should be given to use of dynamic analysis procedures. Even where dynamic analysis is required, the standard does not define in any detail the degree of modeling; an adequate model may have a few dynamic degrees of freedom or tens of thousands of dynamic degrees of freedom. The important point is that the model captures the significant dynamic response features so that the resulting lateral force distribution is valid for design. The designer is responsible to determine whether dynamic analysis is warranted and, if so, the degree of detail required to address adequately the seismic performance.

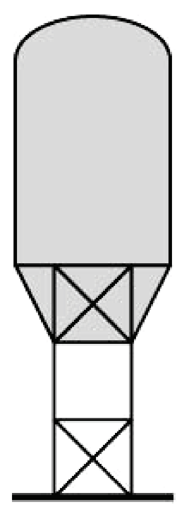

Figure C15.1-8 Soft-story irregularity.

7. Coupled Response: Where the weight of the supported structure is large compared to the weight of the supporting structure, the combined response can be affected significantly by the flexibility of the supported nonbuilding structure. In that case, dynamic analysis of the coupled system is recommended. Examples of such structures are shown in Figure C15.1-9. Part (a) shows a flexible nonbuilding structure with W_p greater than $0.25(W_s + W_p)$, supported by a relatively flexible structure; the flexibility of the supports and attachments should be considered. Part (b) shows flexible equipment connected by a large-diameter, thick-walled pipe and supported by a flexible structure; the structures should be modeled as a coupled system including the pipe.

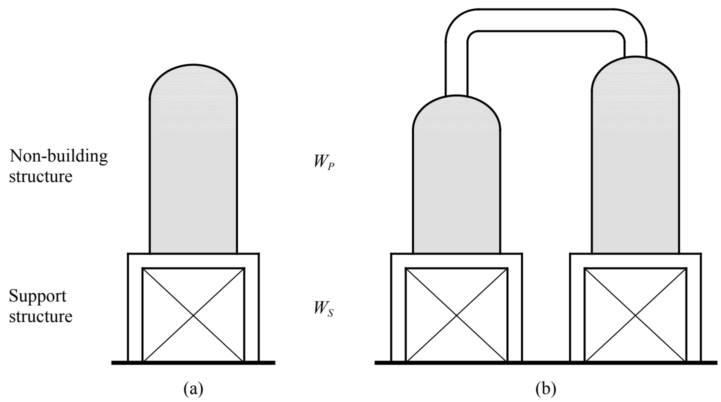

Figure C15.1-9 Coupled system.

C15.2 REFERENCE DOCUMENTS

Chapter 15 of the standard makes extensive use of reference documents in the design of nonbuilding structures for seismic forces. The documents referenced in Chapter 15 are industry documents commonly used to design specific types of nonbuilding structures. The vast majority of these reference documents contain seismic provisions that are based on the seismic ground motions of the 1997 UBC or earlier editions of the UBC. In order to use these reference documents, Chapter 15 modifies the seismic force provisions of these reference documents through the use of "bridging equations." The standard only modifies industry documents that specify seismic demand and capacity. The bridging equations are intended to be used directly with the other provisions of the specific reference documents. Unlike the other provisions of the standard, if the reference documents are written terms of allowable stress design, then the bridging equations are shown in allowable stress design format. In addition, the detailing requirements referenced in Tables 15.4-1 and Table 15.4-2 must be followed, as well as the general requirements found in Section 15.4.1. The usage of reference documents in conjunction with the requirements of Section 15.4.1 are summarized below in Table C15.2-1.

Table C15.2-1 Usage of Reference Documents in Conjunction with Section 15.4.1

Subject	Requirement
R, Ω_0, and C_d values, detailing requirements, and height limits	Use values and limits in Tables 12.2-1, 15.4-1, or 15.4-2 as appropriate. Values from the reference document are not to be used.
Minimum base shear	Use the appropriate value from Equation 15.4-1 or 15.4-2 for nonbuilding structures not similar to buildings. For structures containing liquids, gases, and granular solids supported at the base, the minimum seismic force cannot be less than that required by the reference document.
Importance factor	Use the value from Section 15.4.1.1 based on Occupancy Category. Importance factors from the reference document are not to be used unless they are greater than those provided in the standard.
Vertical distribution of lateral load	Use requirements of Section 12.8.3 or Section 12.9 or the applicable reference document.
Seismic provisions of reference documents	The seismic force provisions of reference documents may be used only if they have the same basis as Section 11.4 and the resulting values for total lateral force and total overturning moment are no less than 80 percent of the values obtained from the standard.
Load combinations	Load combinations specified in Section 2.3 (LRFD) or Section 15 (includes ASD load combinations of Section 2.4) must be used.

Currently, only two reference documents have been revised to meet the seismic requirements of the standard. AWWA D100-05 and API 650 10th Edition Addendum 4 (2005) have been adopted by reference in the standard without modification except that height limits are imposed on "elevated tanks on symmetrically braced legs (not similar to buildings)" in AWWA D100-05. Both of these reference documents apply to welded steel liquid storage tanks.

C15.3 NONBUILDING STRUCTURES SUPPORTED BY OTHER SRUCTURES

There are instances where nonbuilding structures not similar to buildings are supported by other structures or other nonbuilding structures. This section specifies how the seismic design loads for such structures are to be determined and the detailing requirements that are to be satisfied in the design.

C15.3.1 Less than 25 Percent of Combined Weight Condition. In many instances, the weight of the supported nonbuilding structure is relatively small compared to the weight of the supporting structure such that the supported nonbuilding structure will have a relatively small effect on the overall nonlinear earthquake response of the primary structure during design-level ground motions. It is permitted to treat such structures as nonstructural components and use the requirements of Chapter 13 for their design. The ratio of secondary component weight to total weight of 25 percent at which this treatment is permitted is based on judgment and was introduced into code provisions in the 1988 *Uniform Building Code* by the SEAOC Seismology Committee. Analytical studies, typically based on linear elastic primary and secondary structures, indicate that the ratio should be lower, but the SEAOC Seismology Committee judged that the 25 percent ratio is appropriate where primary and secondary structures exhibit non-linear behavior that tends to lessen the effects of resonance and interaction. In cases where a nonbuilding structures (or nonstructural component) is supported by another structure, it may be appropriate to analyze in a single model. In such cases it is intended that seismic design loads and detailing requirements be determined following the procedures of Section 15.3.2. Where there are multiple large nonbuilding structures, such as vessels supported on a primary nonbuilding structure, and the weight of an individual supported nonbuilding structure does not exceed the 25 percent limit but the combined weight of the supported nonbuilding structures does, it is recommended that the combined analysis and design approach of Section 15.3.2 be used. It is also suggested that dynamic analysis be performed in such cases, since the equivalent lateral force procedure may not capture some important response effects in some members of the supporting structure.

Where the weight of the supported nonbuilding structure does not exceed the 25 percent limit and a combined analysis is performed, the following procedure should be used to determine the F_p force of the supported nonbuilding structure based on Equation 13.3-4:

1. A modal analysis should be performed in accordance with Section 12.9. The base shear of the combined structure and nonbuilding structure should be taken as no less than 85 percent of the equivalent lateral force procedure base shear.
2. For a component supported at level i, the acceleration at that level should be taken as a_i, the total shear just below level i divided by the seismic weight at and above level i.
3. The elastic value of the component shear force coefficient should next be determined as the shear force from the modal analysis at the point of attachment of the component to the structure divided by the weight of the component. This value is preliminarily taken as $a_i a_p$. Since a_p cannot be taken as less than 1.0, the value of a_p is taken as $a_i a_p / a_i$, except that the final value a_p need not be taken as greater than 2.5 and should not be taken as less than 1.0. The final value of $a_i a_p$ should be the final value of a_i determined in Step 2 multiplied by the final value of a_p determined earlier in this step.
4. The resulting value of $(a_i a_p)$ should be used in Equation 13.3-4; the resulting value of F_p is subject to the maximum and minimum values of Equations 13.3-2 and 13.3-3, respectively.

C15.3.2 Greater Than or Equal to 25 Percent Combined Weight Condition. Where the weight of the supported structure is relatively large compared to the weight of the supporting structure, the overall response can be affected significantly. The standard sets forth two analysis approaches, depending on the rigidity of the nonbuilding structure. The determination of what is deemed rigid or flexible is based on the same criteria used for nonstructural components.

Where the supported nonbuilding structure is rigid, it is acceptable to treat the supporting structure as a nonbuilding structure similar to a building and to determine its design loads and detailing using the requirements of Section 15.5. The design of the rigid nonbuilding structure and its anchorage is determined using the requirements of Chapter 13 with the amplification factor, a_p, taken as 1.0. However, this is a relatively rare condition since the flexibility of any directly supporting members in the primary structure, such as floor beams, must be considered in determining the period of the component.

In the usual case, where the supported nonbuilding structure is flexible, a combined model of the supporting structure and the supported nonbuilding structure is used. The design loads and detailing are determined based on the lower R value of the supported nonbuilding structure or supporting structure.

Although not specifically mentioned in Section 15.3.2, another approach is permitted. A nonlinear response history analysis of the combined system can be performed in accordance with Section 16.2, and the results can be used for the design of both the supported and supporting nonbuilding structures. This option should be considered where standard static and dynamic elastic analysis approaches may be inadequate to evaluate the earthquake response (such as for suspended boilers). This option should be used with extreme caution since modeling and interpretation of results requires considerable judgment. Due to this sensitivity, Section 16.2 requires independent design review.

C15.4 STRUCTURAL DESIGN REQUIREMENTS

This section specifies the basic coefficients and minimum design forces to be used to determine seismic design loads for nonbuilding structures. It also specifies height limits and restrictions. As with building structures, it presumes that the first step in establishing the design forces is to determine the design base shear for the structure.

There are two types of nonbuilding structures: those with structural systems similar to buildings and those with structural systems not similar to buildings. Specific requirements for these two cases appear in Sections 15.5 and 15.6.

C15.4.1 Design Basis. Separate tables are provided in this section that identify the basic coefficients, associated detailing requirements, and height limits and restrictions for the two types of nonbuilding structures.

For nonbuilding structures similar to buildings, the design seismic loads are determined using the same procedures used for buildings as specified in Chapter 12 with two exceptions: fundamental periods are determined in accordance with Section 15.4.4, and Table 15.4-1 provides additional options for structural systems. Although only Section 12.8 (the equivalent lateral force procedure) is specifically mentioned in Section 15.4.1, Section 15.1.3 provides the analysis procedures that are permitted for nonbuilding structures.

In Table 15.4-1, seismic coefficients, system restrictions, and height limits are specified for a few nonbuilding structures similar to buildings. The values of R, Ω_o, and C_d, the detailing requirement references, and the structural system height limits are the same as those in Table 12.2-1 for the same systems, except for ordinary moment frames. In Chapter 12 increased height limits for ordinary moment frames structural systems apply to metal building systems, while in Chapter 15 they apply to pipe racks with end plate bolted moment connections. The seismic performance of pipe racks was judged to be similar to that of metal building structures with end plate bolted moment connections, so the height limits were made the same as those specified in previous editions.

Table 15.4-1 also provides lower R values with less restrictive height limits in Seismic Design Categories D, E, and F based on good performance in past earthquakes. For some options, no seismic detailing is required if very low values of R (and corresponding high seismic design forces) are used. The concept of extending this approach to other structural systems is the subject of future research using the methodology developed by the ATC 63 project.

For nonbuilding structures not similar to buildings, the seismic design loads are determined as in Chapter 12 with three exceptions: the fundamental periods are determined in accordance with Section 15.4.4, the minima are those specified in Section 15.4.1.2, and the seismic coefficients are those specified in Table 15.4-2.

Some entries in Table 15.4-2 may seem to be conflicting or confusing. For example, the first major entry is for elevated tanks, vessels, bins, or hoppers. A subset of this entry is for tanks on braced or unbraced legs. This subentry is intended for structures where the supporting columns are integral with the shell (such as an elevated water tank). Tension-only bracing is allowed for such a structure. Where the tank or vessel is supported by building-like frames, the frames are to be designed in accordance with all of the restrictions normally applied to building frames. The entry for tanks or vessels supported on structural towers similar to buildings assumes that the operating weight of the supported tank or vessel is less than 25 percent of the total weight; if the ratio is greater than 25 percent, the proper entry is that most closely related to the subject vessel or bin.

C15.4.1.1 Importance Factor. The importance factor for a nonbuilding structure is based on the occupancy category defined in Chapter 1 of the standard or the building code being used in conjunction with the standard. In some cases, reference standards provide a higher importance factor, in which case the higher importance factor is used.

If the importance factor is taken as 1.0 based on a Hazard and Operability (HAZOP) analysis performed in accordance with Chapter 1, the third paragraph of Section 1.5.2 requires careful consideration; worst-case scenarios (instantaneous release of a vessel or piping system) must be considered. HAZOP risk analysis consultants often do not make such assumptions, so the design professional should review the HAZOP analysis with the HAZOP consultant to confirm that such assumptions have been made in order to validate adjustment of the importance factor. Clients may not be aware that HAZOP consultants do not normally consider the worst-case scenario of instantaneous release but tend to focus on other more hypothetical limited-release scenarios, such as those associated with a 2-inch square hole in a tank or vessel.

C15.4.2 Rigid Nonbuilding Structures. The definition of rigid (having a natural period of less than 0.06 second) was selected judgmentally. Below that period, the energy content of seismic ground motion is generally believed to be very low, and therefore the building response is not likely to be excessively amplified. Also, it is unlikely that any building will have a first mode period as low as 0.06 second, and it is even unusual for a second mode period to be that low. Thus, the likelihood of either resonant behavior or excessive amplification becomes quite small for equipment having periods below 0.06 second.

The analysis to determine the period of the nonbuilding structure should include the flexibility of the soil subgrade.

C15.4.3 Loads. As for buildings, the seismic weight must include the range of design operating weight of permanent equipment.

C15.4.4 Fundamental Period. A significant difference between building structures and nonbuilding structures is that the approximate period formulas and limits of Section 12.8.2.1 may not be used for nonbuilding structures. In lieu of calculating a specific period for a nonbuilding structure for determining seismic lateral forces, it is of course conservative to assume a period of T_s (= S_{D1}/S_{DS}) which results in the largest lateral design forces. Computing the fundamental period is not considered a significant burden, since most commonly used computer analysis programs can perform the required calculations.

C15.4.8 Site-Specific Response Spectra. Where site-specific response spectra are required, they should be developed in accordance with Chapter 21 of the standard. If determined for other recurrence intervals, Section 21.1 applies, but Sections 21.2 through 21.4 apply only to MCE determinations. Where other recurrence intervals are used, it should be demonstrated that the requirements of Chapter 15 also are satisfied.

C15.5 NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

C15.5.1 General. Although certain nonbuilding structures exhibit behavior similar to that of building structures, their functions and occupancies are different. Section 15.5 of the standard addresses the differences.

C15.5.2 Pipe Racks. Free-standing pipe racks supported at or below grade with framing systems that are similar to building systems are designed in accordance with Section 12.8 or 12.9 and Section 15.4. Single-column pipe racks that resist lateral loads should be designed as inverted pendulums.

Based on good performance in past earthquakes, Table 15.4-1 sets forth the option of lower *R* values and less restrictive height limits for structural systems commonly used in pipe racks. The *R* value versus height limit trade-off recognizes that the size of some nonbuilding structures is determined by factors other than traditional loadings and results in structures that are much stronger than required for seismic loadings. Therefore, the ductility demand is generally much lower than that for a corresponding building. The intent is to obtain the same structural performance at the increased heights. This option will prove to be economical in most situations due to the relative cost of materials and construction labor. The lower *R* values and increased height limits of Table 15.4-1 apply to nonbuilding structures similar to buildings; they cannot be applied to building structures. Table C15.5-1 illustrates the *R* values and height limits for a 70-foot-high steel ordinary moment frame (OMF) pipe rack.

Table C15.5-1 *R* Value Selection Example for Steel OMF Pipe Racks

SDC	*R*	ASCE/SEI 7-05 Table	System	Seismic Detailing Requirements
C	3.5	12.2-1 or 15.4-1	Ordinary steel moment frame	AISC 341
C	3	12.2-1	Structural steel systems not specifically detailed for seismic resistance	None
D or E	2.5	15.4-1	Steel OMF with permitted height increase	AISC 341 (AISC Seismic)
D, E, or F	1	15.4-1	Steel OMF with unlimited height	None

C15.5.3 Steel Storage Racks. The two approaches to the design of steel storage racks set forth by the standard are intended to produce comparable results.

These recommendations address the concern that storage racks in warehouse-type retail stores may pose a greater seismic risk to the general public than exists in low-occupancy warehouses or more conventional retail environments. Under normal conditions, retail stores have a far higher occupant load than an ordinary warehouse of a comparable size. Failure of a storage rack system in a retail environment is much more likely to cause personal injury than a similar failure in a storage warehouse. To provide an appropriate level of additional safety in areas open to the public, an importance factor of 1.50 is specified. Storage rack contents, while beyond the scope of the standard, may pose a potentially serious threat to life should they fall from the shelves in an earthquake. It is recommended that restraints be provided, as shown in Figure C15.5-1, to prevent the contents of rack shelving open to the general public from falling during strong ground shaking.

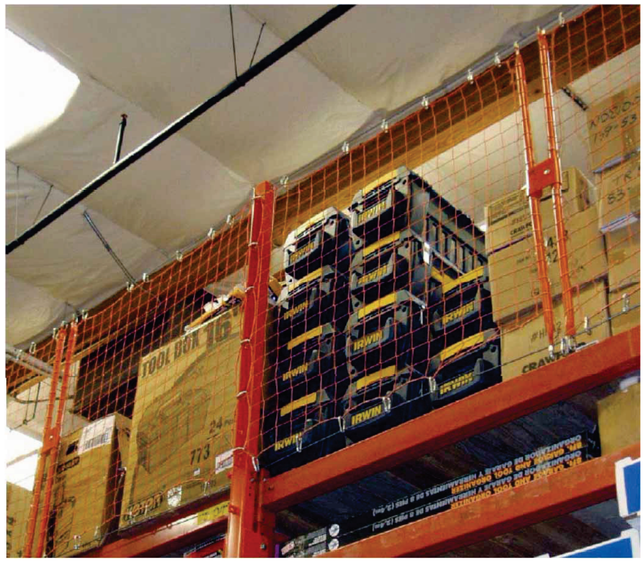

Figure C15.5-1 Merchandise restrained by netting.

C15.5.4 Electrical Power Generating Facilities. Electrical power plants closely resemble building structures, and their performance in seismic events has been good. For reasons of mechanical performance, lateral drift of the structure must be limited. The lateral bracing system of choice has been the concentrically braced frame. In the past, the height limits on braced frames in particular have been an encumbrance to the design of large power generating facilities. Based on acceptable past performance, Table 15.4-1 permits the use of CBRs with both lower *R* values and less restrictive height limits. This option is particularly effective for boiler buildings that generally are 300 feet or more in height. A peculiarity of large boiler buildings is the general practice of suspending the boiler from the roof structures; this results in an unusual mass distribution as discussed in Section C15.1.3.

C15.5.5 Structural Towers for Tanks and Vessels. The requirements of this section apply to structural towers that are not integral with the supported tank. Elevated water tanks designed in accordance with AWWA D100-06 are not subject to Section 15.5.5.

C15.5.6 Piers and Wharves. Current industry practice recognizes the distinct differences between the two categories of piers and wharves described in the standard. Piers and wharves with public occupancy, described in Section 15.5.6.2, are commonly treated as the "foundation" for buildings or building-like structures; design is performed using the standard, likely under the jurisdiction of the local building official. Piers and wharves without occupancy by the general public are often treated differently and are outside the scope of the standard; in many cases, these structures do not fall under the jurisdiction of building officials, and design is performed using other industry-accepted approaches.

Design decisions associated with these structures often reflect economic considerations by both owners and local, regional, or state jurisdictional entities with interest in commercial development. Where building officials have jurisdiction but lack experience analyzing pier and wharf structures, reliance on other industry-accepted design approaches is common.

Where occupancy by the general public is not a consideration, seismic design of structures at major ports and marine terminals often uses a performance-based approach, with criteria and methods that are very different from those used for buildings, as provided in the standard. Design approaches most commonly used are generally consistent with the practices and criteria described in the following documents:

1. *Seismic Design Guidelines for Port Structures*, Working Group No. 34 of the Maritime Navigation Commission (PIANC/MarCom/WG34), A. A. Balkema, Lisse, Netherlands, 2001.
2. *Seismic Criteria for California Marine Oil Terminals,* Vol. 1 and Vol. 2, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Ferritto, J., Dickenson, S., Priestley N., Werner, S., Taylor, C., Burke D., Seelig

W., and Kelly, S., Port Hueneme, CA, 1999.

3. *Seismic Design and Retrofit of Bridges*, Priestley, N.J.N., Siebel, F., and Calvi, G.M., New York, 1996.
4. *Seismic Guidelines for Ports*, by the Ports Committee of the Technical Council on Lifeline Earthquake Engineering, ASCE/SEI, edited by Stuart D. Werner, Monograph No. 12, published by ASCE, Reston, Virginia, March 1998.
5. *MOTEMS, 2005, "Marine Oil Terminal Engineering and Maintenance Standards"*, 2001 Title 24, Part 2, California Building Code, Chapter 31F*,* January 31, 2005.

These alternative approaches have been developed over a period of many years by working groups within the industry, and they reflect the historical experience and performance characteristics of these structures, which are very different from those of building structures.

The main emphasis of the performance-based design approach is to provide criteria and methods that depend on the economic importance of a facility. Adherence to the performance criteria in the documents listed above does not seek to provide uniform margins of collapse for all structures; their application is expected to provide at least as much inherent life-safety as for buildings designed using the standard. The reasons for the higher inherent level of life-safety for these structures include the following:

1. These structures have relatively infrequent occupancy, with few working personnel and very low density of personnel. Most of these structures consist primarily of open area, with no enclosed structures that can collapse onto personnel. Small control buildings on marine oil terminals or similar secondary structures are commonly designed in accordance with the local building code.
2. These pier or wharf structures typically are constructed of reinforced concrete, prestressed concrete, or steel and are highly redundant due to the large number of piles supporting a single wharf deck unit. Tests done at the University of California at San Diego for the Port of Los Angeles have shown that very high ductilities (10 or more) can be achieved in the design of these structures using practices currently employed in California ports.
3. Container cranes, loading arms, and other major structures or equipment on piers or wharves are specifically designed not to collapse in an earthquake. Typically, additional piles and structural members are incorporated into the wharf or pier specifically to support such items.
4. Experience has shown that seismic "failure" of wharf structures in zones of strong seismicity is indicated not by collapse but by economically irreparable deformations of the piles. The wharf deck generally remains level or slightly tilting, but has shifted out of position. Complete failure that could endanger life-safety due to earthquake loading has never occurred historically where the structure in the marine environment has been maintained properly.
5. The performance-based criteria of the listed documents address reparability of the structure, which is much more stringent criteria than collapse prevention and results in a greater margin for life-safety.

Lateral load design of these structures in low, or even moderate, seismic regions often is governed by other marine conditions.

C15.6 GENERAL REQUIREMENTS FOR NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding structures not similar to buildings exhibit behavior markedly different from that of building structures. Most of these types of structures have reference documents that address their unique structural performance and behavior. The ground motion in the standard requires appropriate translation to allow use with industry standards.

C15.6.1 Earth-Retaining Structures. Section C11.8.3 presents commonly used approaches for the design of nonyielding walls and yielding walls for bending, overturning, sliding, etc., taking into account the varying soil types, importance, and site seismicity.

C15.6.2 Stacks and Chimneys. The design of stacks and chimneys to resist natural hazards generally is governed by wind design considerations. The exceptions to this general rule involve locations with high seismicity, stacks and chimneys with large elevated masses, and stacks and chimneys with unusual geometries. It is prudent to evaluate the effect of seismic loads in all but those areas with the lowest seismicity. Although not specifically required, it is recommended that the special seismic details required elsewhere in the standard be considered for application to stacks and chimneys.

Guyed steel stacks and chimneys generally are lightweight. As a result, the design loads due to natural hazards generally are governed by wind. On occasion, large flares or other elevated masses located near the top may require in-depth seismic analysis. Although it does not specifically address seismic loading, Chapter 6 of Troitsky (1982) provides a methodology appropriate for resolution of the seismic forces defined in the standard.

C15.6.4 Special Hydraulic Structures. The most common special hydraulic structures are baffle walls and weirs that are used in water treatment and waste water treatment plants. Because there are openings in the walls, during normal operations the fluid levels are equal on each side of the wall, exerting no net horizontal force. Sloshing during a seismic event can exert large forces on the wall, as illustrated in Figure C15.6-1. The walls can fail unless they are designed properly to resist the dynamic fluid forces.

C15.6.5 Secondary Containment Systems. This section reflects the judgment that designing all impoundment dikes for the MCE ground motion when full and sizing all impoundment dikes for the sloshing wave is too conservative. Designing an impoundment dike as full for the MCE assumes failure of the primary containment and occurrence of a significant aftershock. Such significant aftershocks (of the same magnitude as the MCE ground motion) are rare and do not occur in all locations. While explicit design for aftershocks is not a requirement of the standard, secondary containment must be designed full for an aftershock to protect the general public. The use of two-thirds of the MCE ground motion as the magnitude of the design aftershock is supported by Bath's Law, according to which the maximum expected aftershock magnitude may be estimated to be 1.2 scale units below the main shock magnitude.

The risk assessment and risk management plan described in Section 1.5.2 are used to determine where the secondary containment must be designed full for the MCE. The decision to design secondary containment for this more severe condition should be based on the likelihood of a significant aftershock occurring at the particular site, considering the risk posed to the general public by the release of hazardous material from the secondary containment.

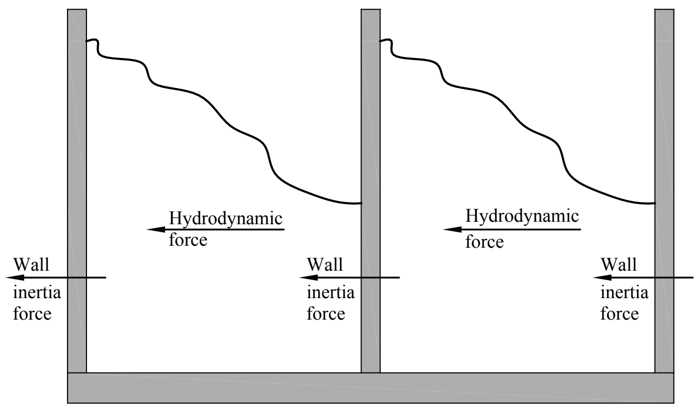

Figure C15.6-1 Wall forces.

Secondary containment systems must be designed to contain the sloshing wave where the release of liquid would place the general public at risk by exposing them to hazardous materials, by scouring of foundations of adjacent structures, or by causing other damage to adjacent structures.

C15.6.6 Telecommunication Towers. Telecommunication towers support small masses, and their design generally is governed by wind forces. Although telecommunication towers have a history of experiencing seismic events without failure or significant damage, seismic design in accordance with the standard is required.

Typically bracing elements bolt directly (without gusset plates) to the tower legs, which consist of pipes or bent plates in a triangular plan configuration.

C15.7 TANKS AND VESSELS

C15.7.1 General. Methods for seismic design of tanks, currently adopted by a number of reference documents, have evolved from earlier analytical work by Jacobsen, Housner, Veletsos, Haroun, and others. The procedures used to design flat-bottom storage tanks and liquid containers are based on the work of Housner, Wozniak, and Mitchell. The reference documents for tanks and vessels have specific requirements to safeguard against catastrophic failure of the primary structure based on observed behavior in seismic events since the 1930s. Other methods of analysis, using flexible shell models, have been proposed but at present are beyond the scope of the standard.

The industry-accepted design methods employ three basic steps:

1. Dynamic modeling of the structure and its contents. When a liquid-filled tank is subjected to ground acceleration, the lower portion of the contained liquid, identified as the impulsive component of mass, W_i, acts as if it were a solid mass

rigidly attached to the tank wall. As this mass accelerates, it exerts a horizontal force, P_i, on the wall; this force is directly proportional to the maximum acceleration of the tank base. This force is superimposed on the inertia force of the accelerating wall itself, P_s. Under the influence of the same ground acceleration, the upper portion of the contained liquid responds as if it were a solid mass flexibly attached to the tank wall. This portion, which oscillates at its own natural frequency, is identified as the convective component, W_c, and exerts a horizontal force, P_c, on the wall. The convective component oscillations are characterized by sloshing whereby the liquid surface rises above the static level on one side of the tank and drops below that level on the other side.

2. Determination of the period of vibration, T_i, of the tank structure and the impulsive component; and determination of the natural period of oscillation (sloshing), T_c, of the convective component.
3. Selection of the design response spectrum. The response spectrum may be site-specific or it may be constructed on the basis of seismic coefficients given in national codes and standards. Once the design response spectrum is constructed, the spectral accelerations corresponding to T_i and T_c are obtained and are used to calculate the dynamic forces P_i, P_s, and P_c.

Detailed guidelines for the seismic design of circular tanks, incorporating these concepts to varying degrees, have been the province of at least four industry reference documents: AWWA D100 for welded steel tanks (since 1964); API 650 for petroleum storage tanks; AWWA D110 for prestressed, wire-wrapped tanks (since 1986); and AWWA D115 for prestressed concrete tanks stressed with tendons (since 1995). In addition, API 650 and API 620 contain provisions for petroleum, petrochemical, and cryogenic storage tanks. The detail and rigor of analysis prescribed in these documents have evolved from a semi-static approach in the early editions to a more rigorous approach at present, reflecting the need to include the dynamic properties of these structures.

The requirements in Section 15.7 are intended to link the latest procedures for determining design-level seismic loads with the allowable stress design procedures based on the methods in the standard. These requirements, which in many cases identify specific substitutions to be made in the design equations of the reference documents, will assist users of the standard in making consistent interpretations.

ACI has published ACI 350.3-01, "*Seismic Design of Liquid-Containing Concrete Structures.*" This document, which addresses all types of concrete tanks (prestressed and non-prestressed, circular and rectilinear), has provisions that are consistent with the seismic criteria of the 2000 *Provisions.* The document serves as both a practical "ho-to" loading reference and a guide to supplement application of ACI 318 Chapter 21.

C15.7.2 Design Basis. In the case of the seismic design of nonbuilding structures, standardization requires adjustments to industry reference documents to minimize existing inconsistencies among them, while recognizing that structures designed and built over the years in accordance with these documents have performed well in earthquakes of varying severity. Of the inconsistencies among reference documents, the ones most important to seismic design relate to the base shear equation. The traditional base shear takes the following form:

$$V = \frac{ZIS}{R_w} CW \qquad \text{(C15.7-1)}$$

An examination of those terms as used in the different references reveals the following:

1. *ZS:* The seismic zone coefficient, *Z*, has been rather consistent among all the documents since it usually has been obtained from the seismic zone designations and maps in the model building codes. On the other hand, the soil profile coefficient, *S*, does vary from one document to another. In some documents these two terms are combined.
2. *I*: The importance factor, *I*, has varied from one document to another, but this variation is unavoidable and understandable owing to the multitude of uses and degrees of importance of tanks and vessels.
3. *C:* The coefficient *C* represents the dynamic amplification factor that defines the shape of the design response spectrum for any given ground acceleration. Since *C* is primarily a function of the frequency of vibration, inconsistencies in its derivation from one document to another stem from at least two sources: differences in the equations for the determination of the natural frequency of vibration, and differences in the equation for the coefficient itself. (For example, for the shell/impulsive liquid component of lateral force, the steel tank documents use a constant design spectral acceleration [constant *C*] that is independent of the "impulsive" period, *T*.) In addition, the value of *C* will vary depending on the damping ratio assumed for the vibrating structure (usually between 2 percent and 7 percent of critical).
4. Where a site-specific response spectrum is available, calculation of the coefficient *C* is not necessary except in the case of the convective component (coefficient C_c) which is assumed to oscillate with 0.5 percent of critical damping and whose period of oscillation is usually long (greater than 2.5 seconds). Since site-specific spectra are usually constructed for high damping values (3 percent to 7 percent of critical) and since the site-specific spectral profile may not be well-

defined in the long-period range, an equation for C_c applicable to a 0.5 percent damping ratio is necessary in order to calculate the convective component of the seismic force.

5. R_w: The response modification factor, R_w, is perhaps the most difficult to quantify, for a number of reasons. While R_w is a compound coefficient that is supposed to reflect the ductility, energy-dissipating capacity, and redundancy of the structure, it is also influenced by serviceability considerations, particularly in the case of liquid-containing structures.

In the standard the base shear equation for most structures has been reduced to $V = C_s W$, where the seismic response coefficient, C_s, replaces the product ZSC/R_w. C_s is determined from the design spectral response acceleration parameters S_{DS} and S_{D1} (at short periods and at a period of 1, respectively) which, in turn, are obtained from the mapped MCE spectral accelerations S_s and S_1. As in the case of the prevailing industry reference documents, where a site-specific response spectrum is available, C_s is replaced by the actual values of that spectrum.

The standard contains several bridging equations, each designed to allow proper application of the design criteria of a particular reference document in the context of the standard. These bridging equations associated with particular types of liquid-containing structures and the corresponding reference documents are discussed below. Calculation of the periods of vibration of the impulsive and convective components is in accordance with the reference documents, and the detailed resistance and allowable stresses for structural elements of each industry structure are unchanged, except where new information has led to additional requirements.

It is expected that the bridging equations of Sections 15.7.7.3 and 15.7.10.7 will be eliminated as the relevant reference documents are updated to conform to the standard. The bridging equations previously provided for AWWA D100 and API 650 already have been eliminated as a result of updates of these documents.

C15.7.3 Strength and Ductility. As is the case for building structures, ductility and redundancy in the lateral support systems for tanks and vessels are desirable and necessary for good seismic performance. Tanks and vessels are not highly redundant structural systems, and therefore ductile materials and well-designed connection details are needed to increase the capacity of the vessel to absorb more energy without failure. The critical performance of many tanks and vessels is governed by shell stability requirements rather than by yielding of the structural elements. For example, contrary to building structures, ductile stretching of anchor bolts is a desirable energy absorption component where tanks and vessels are anchored. The performance of cross-braced towers is highly dependent on the ability of the horizontal compression struts and connection details to develop fully the tension yielding in the rods. In such cases, it is also important to preclude both premature failure in the threaded portion of the connection and failure of the connection of the rod to the column prior to yielding of the rod.

C15.7.4 Flexibility of Piping Attachments. Poor performance of piping connections (tank leakage and damage) due to seismic deformations is a primary weakness observed in recent seismic events. While commonly used piping connections can impart mechanical loads to the tank shell, proper design in seismic areas results in only negligible mechanical loads on tank connections subject to the displacements shown in Table 15.7-1. API 650 treats the values shown in Table 15.7-1 as allowable stress based values and therefore requires that these values be multiplied by 1.4 where strength-based capacity values are required for design.

In addition, interconnected equipment, walkways, and bridging between multiple tanks must be designed to resist the loads and accommodate the displacements imposed by seismic forces. Unless connected tanks and vessels are founded on a common rigid foundation, the calculated differential movements must be assumed to be out of phase.

C15.7.5 Anchorage. Many steel tanks can be designed without anchors by using annular plate detailing in accordance with reference documents. Where tanks must be anchored due to overturning potential, proper anchorage design will provide both a shell attachment and an embedment detail that will yield the bolt without tearing the shell or pulling the bolt out of the foundation. Properly designed anchored tanks have greater reserve strength to resist seismic overload than do unanchored tanks.

Where anchor bolts and attachments are misaligned such that the anchor nut or washer does not bear evenly on the attachment, additional bending stresses in threaded areas may cause premature failure before anchor yielding.

C15.7.6 Ground-Supported Storage Tanks for Liquids

C15.7.6.1 General. The response of ground storage tanks to earthquakes is well documented by Housner, Mitchell and Wozniak, Veletsos, and others. Unlike building structures, the structural response of these tanks is influenced strongly by the fluid-structure interaction. Fluid-structure interaction forces are categorized as sloshing (convective) and rigid (impulsive) forces. The proportion of these forces depends on the geometry (height-to-diameter ratio) of the tank. API 650, API 620, AWWA D100, AWWA D110, AWWA D115, and ACI 350.3 provide the data necessary to determine the relative masses and moments for each of these contributions.

The standard requires that these structures be designed in accordance with the prevailing reference documents, except that the height of the sloshing wave, δ_s, must be calculated using Equations 15.7-13. Note that API 650 and AWWA D100 include this requirement in their latest editions.

Equations 15.7-10 and 15.7-11 provide the spectral acceleration of the sloshing liquid for the constant-velocity and constant-displacement regions of the response spectrum, respectively. The 1.5 factor in these equations is an adjustment for 0.5 percent damping.

Small-diameter tanks and vessels are more susceptible to overturning and vertical buckling. As a general rule, a greater ratio of *H*/*D* produces lower resistance to vertical buckling. Where *H*/*D* is greater than 2, overturning approaches "rigid mass" behavior (the sloshing mass is small). Large-diameter tanks may be governed by additional hydrodynamic hoop stresses in the middle regions of the shell.

The impulsive period (the natural period of the tank components and the impulsive component of the liquid) is typically in the 0.25 to 0.6 second range. Many methods are available for calculating the impulsive period. The Veletsos flexible-shell method is commonly used by many tank designers. For example, see Velestos (1974) and Malhotra, Wenk, and Wieland (2000).

C15.7.6.1.1 Distribution of Hydrodynamic and Inertia Forces. Most of the reference documents for tanks define reaction loads at the base of shell-foundation interface, without indicating the distribution of loads on the shell as a function of height. ACI 350.3 specifies the vertical and horizontal distribution of such loads.

The overturning moment at the base of the shell in the industry reference documents is only the portion of the moment that is transferred to the shell. The total overturning moment also includes the variation in bottom pressure, which is an important consideration for design of pile caps, slabs, or other support elements that must resist the total overturning moment. Wozniak and Mitchell (1978) and TID 7024 (1963) provide additional information.

C15.7.6.1.2 Sloshing. In past earthquakes, sloshing contents in ground storage tanks has caused both leakage and non-catastrophic damage to the roof and internal components. Even this limited damage, and the associated costs and inconvenience, can be significantly mitigated where the following items are considered:

1. Effective masses and hydrodynamic forces in the container.
2. Impulsive and pressure loads at
    a. The sloshing zone (that is, the upper shell and edge of the roof system),
    b. The internal supports (such as roof support columns and tray-supports), and
    c. The internal equipment (such as distribution rings, access tubes, pump wells, and risers).
3. Freeboard (which depends on the sloshing wave height).

A minimum freeboard of $0.7\delta_s$ is recommended for economic considerations but is not required.

Tanks and vessels storing biologically or environmentally benign materials typically do not require freeboard to protect the public health and safety. However, providing freeboard in areas of frequent seismic occurrence for vessels normally operated at or near top capacity may lessen damage (and the cost of subsequent repairs) to the roof and upper container.

The sloshing wave height specified in Section 15.7.6.1.2 is based on the design earthquake defined in the standard. For economic reasons, freeboard for tanks assigned to Occupancy Category I, II, or III may be calculated using a fixed value of T_L equal to 4 seconds (as indicated in Section 15.7.6.1, Note d) but using the appropriate importance factor taken from Table 11.5-1. Due to life-safety concerns, freeboard for tanks assigned to Occupancy Category IV must be based on the mapped value of T_L. Because use of the mapped value of T_L results in the theoretical maximum value of freeboard, the calculation of freeboard in the case of Occupancy Category IV tanks is based on an importance factor equal to 1.0 (as indicated in Section 15.7.6.1, Note c).

If the freeboard provided is less than the computed sloshing height, δ_s, the sloshing liquid will impinge on the roof in the vicinity of the roof-to-wall joint, subjecting it to a hydrodynamic force. This force may be approximated by considering the sloshing wave as a hypothetical static liquid column having a height, δ_s. The pressure exerted at any point along the roof at a distance y_s above the at-rest surface of the stored liquid may be assumed equal to the hydrostatic pressure exerted by the hypothetical liquid column at a distance $\delta_s - y_s$ from the top of that column. A better approximation of the pressure exerted on the roof is found in Malhotra (2005 and 2006).

Another effect of a less-than-full freeboard is that the restricted convective (sloshing) mass "converts" into an impulsive mass thus increasing the impulsive forces. This effect should be taken into account in the tank design. A method for converting the restricted convective mass into an impulsive mass is found in Malhotra (2005 and 2006). It is recommended that sufficient freeboard to accommodate the full sloshing height be provided wherever possible.

C15.7.6.1.4 Internal Components. Wozniak and Mitchell (1978) provides a recognized analysis method for determining the lateral loads on internal components due to sloshing liquid.

C15.7.6.1.5 Sliding Resistance. Historically, steel ground-supported tanks full of product have not slid off foundations. A few unanchored, empty tanks or bulk storage tanks without steel bottoms have moved laterally during earthquake ground shaking. In most cases, these tanks may be returned to their proper locations. Resistance to sliding is obtained from the frictional resistance between the steel bottom and the sand cushion on which bottoms are placed. Because tank bottoms usually are crowned upward toward the tank center and are constructed of overlapping, fillet-welded, individual steel plates (resulting in a rough bottom), it is reasonably conservative to take the ultimate coefficient of friction as 0.70 (AISC, 1986), and therefore a value of tan 30^{o} (= 0.577) is used in design. The value of 30^{o} represents the internal angle of friction of sand. The vertical weight of the tank and contents, as reduced by the component of vertical acceleration, provides the net vertical load. An orthogonal combination of vertical and horizontal seismic forces, following the procedure in Section 12.5.3, may be used.

C15.7.6.1.6 Local Shear Transfer. The transfer of seismic shear from the roof to the shell and from the shell to the base is accomplished by a combination of membrane shear and radial shear in the wall of the tank. For steel tanks, the radial (out-of-plane) seismic shear is very small and usually is neglected; thus, the shear is assumed to be resisted totally by membrane (in-plane) shear. For concrete walls and shells, which have a greater radial shear stiffness, the shear transfer may be shared. The ACI 350.3 commentary provides further discussion.

C15.7.6.1.7 Pressure Stability. Internal pressure may increase the critical buckling capacity of a shell. Provision to include pressure stability in determining the buckling resistance of the shell for overturning loads is included in AWWA D100. Recent testing on conical and cylindrical shells with internal pressure yielded a design methodology for resisting permanent loads in addition to temporary wind and seismic loads. See Miller, Meier, and Czaska (1997).

C15.7.6.1.8 Shell Support. Anchored steel tanks should be shimmed and grouted to provide proper support for the shell and to reduce impact on the anchor bolts under reversible loads. The high bearing pressures on the toe of the tank shell may cause inelastic deformations in compressible material (such as fiberboard), creating a gap between the anchor and the attachment. As the load reverses, the bolt is no longer snug and an impact of the attachment on the anchor can occur. Grout is a structural element and should be installed and inspected as an important part of the vertical- and lateral-force-resisting system.

C15.7.6.1.9 Repair, Alteration, or Reconstruction. During their service life, storage tanks are frequently repaired, modified, or relocated. Repairs often are related to corrosion, improper operation, or overload from wind or seismic events. Modifications are made for changes in service, updates to safety equipment for changing regulations, or installation of additional process piping connections. It is imperative these repairs and modifications be designed and implemented properly to maintain the structural integrity of the tank or vessel for seismic loads as well as the design operating loads.

The petroleum steel tank industry has developed specific guidelines in API 653 that are statutory requirements in some states. It is recommended that the provisions of API 653 also be applied to other liquid storage tanks (water, wastewater, chemical, etc.) as it relates to repairs, modifications, or relocation that affects the pressure boundary or lateral force-resisting system of the tank or vessel.

C15.7.7 Water Storage and Water Treatment Tanks and Vessels. The AWWA design requirements for ground-supported steel water storage structures use allowable stress design procedures that conform to the requirements of the standard.

C15.7.8 Petrochemical and Industrial Tanks and Vessels Storing Liquids

C15.7.8.1 Welded Steel. The American Petroleum Institute (API) uses an allowable stress design procedure that conforms to the requirements of the standard.

The most common damage to tanks observed during past earthquakes includes the following:

1. Buckling of the tank shell near the base due to excessive axial membrane forces. This buckling damage is usually evident as “elephant foot” buckles a short distance above the base or as diamond-shaped buckles in the lower ring. Buckling of the upper ring also has been observed.
2. Damage to the roof due to impingement on the underside of the roof of sloshing liquid with insufficient freeboard.
3. Failure of piping or other attachments that are overly restrained.
4. Foundation failures.

Other than the above damage, the seismic performance of floating roofs during earthquakes has generally been good, with damage usually confined to the rim seals, gage poles, and ladders. However, floating roofs have sunk in some earthquakes due to lack of adequate freeboard or the proper buoyancy and strength required by API 650. Similarly the performance of open tops with top wind girder stiffeners designed per API 650 has been generally good.

C15.7.8.2 Bolted Steel. Bolted steel tanks are often used for temporary functions. Where use is temporary, it may be acceptable to the jurisdictional authority to design bolted steel tanks for no seismic loads or for reduced seismic loads based on a reduced return period. For such reduced loads based on reduced exposure time, the owner should include a signed removal contract with the fixed removal date as part of the submittal to the authority having jurisdiction.

C15.7.9 Ground-Supported Storage Tanks for Granular Materials

C15.7.9.1 General. The response of a ground-supported storage tank storing granular materials to a seismic event is highly dependent on its height-to-diameter (*H*/*D*) ratio and the characteristics of the stored product. The effects of intergranular friction are described in more detail in C15.7.9.3.1 (increased lateral pressure), C15.7.9.3.2 (effective mass), and C15.7.9.3.3 (effective density).

Long-term increases in shell hoop tension due to temperature changes after the product has been compacted also must be included in the analysis of the shell; Anderson (1966) provides a suitable method.

C15.7.9.2 Lateral Force Determination. Seismic forces acting on ground-supported liquid storage tanks are divided between impulsive and convective (sloshing) components. However, in a ground-supported storage tank for granular materials, all seismic forces are of the impulsive type and relate to the period of the storage tank itself. Due to the relatively short period of a tank shell, the response is normally in the constant acceleration region of the response spectrum, which relates to S_{DS}. Therefore, the seismic base shear is calculated as follows:

$$V = \frac{S_{DS}}{\left(\frac{R}{I}\right)} W_{Effective} \tag{C15.7-2}$$

where V, S_{DS}, I, and R have been previously defined, and $W_{Effective}$ is the gross weight of the stored product multiplied by an effective mass factor and an effective density factor, as described in Sections C15.7.9.3.2 and C15.7.9.3.3, plus the dead weight of the tank. Unless substantiated by testing, it is recommended that the product of the effective mass factor and the effective density factor be taken as no less than 0.5 due to the limited test data and the highly variable properties of the stored product.

C15.7.9.3 Force Distribution to Shell and Foundation

C15.7.9.3.1 Increased Lateral Pressure. In a ground-supported tank storing granular materials, increased lateral pressures develop as a result of rigid body forces that are proportional to ground acceleration. Information concerning design for such pressure is scarce. Trahair et al. (1983) describes both a very simple, conservative method and a very difficult, analytical method using failure wedges based on the Mononobe-Okabe modifications of the classical Coulomb method.

C15.7.9.3.2 Effective Mass. For ground-supported tanks storing granular materials, much of the lateral seismic load can be transferred directly into the foundation, via intergranular shear, before it can reach the tank shell. The effective mass that loads the tank shell is highly dependent on the *H*/*D* ratio of the tank and the characteristics of the stored product. Quantitative information concerning this effect is scarce, but Trahair et al. (1983) describes a very simple, conservative method to determine the effective mass. That method presents reductions in effective mass, which may be significant, for *H*/*D* ratios less than 2. This effect is absent for elevated tanks.

C15.7.9.3.3 Effective Density. Granular material stored in tanks (both ground-supported and elevated) does not behave as a solid mass. Energy loss through intergranular movement and grain-to-grain friction in the stored material effectively reduces the mass subject to horizontal acceleration. This effect may be quantified by an effective density factor less than 1.0.

Based on Chandrasekaran and Jain (1968) and on shake-table tests reported in Chandrasekaran et al. (1968), ACI 313 recommends an effective density factor of not less than 0.8 for most granular materials. According to Chandrasekaran and Jain (1968), an effective density factor of 0.9 is more appropriate for materials with high moduli of elasticity, such as aggregates and metal ores.

C15.7.9.3.4 Lateral Sliding. Most ground-supported steel storage tanks for granular materials rest on a base ring and do not have a steel bottom. To resist seismic base shear, a partial bottom or annular plate is used in combination with anchor bolts or a curb angle. An annular plate can be used alone to resist the seismic base shear through friction between the plate and the foundation, in which case the friction limits of Section 15.7.6.1.5 apply. The curb angle detail serves to keep the base of the shell round while allowing it to move and flex under seismic load. Various base details are shown in Figure 13 of Kaups and Lieb (1985).

C15.7.9.3.5 Combined Anchorage Systems. This section is intended to apply to combined anchorage systems that share loads based on their relative stiffnesses, and not to systems where sliding is resisted completely by one system (such as a steel annular plate) and overturning is resisted completely by another system (such as anchor bolts).

C15.7.10 Elevated Tanks and Vessels for Liquids and Granular Materials

C15.7.10.1 General. The three basic lateral load-resisting systems for elevated water tanks are defined by their support structure:

1. Multi-leg braced steel tanks (trussed towers, as shown in Figure C15.7-1),
2. Small-diameter single-pedestal steel tanks (cantilever columns, as shown in Figure C15.7-2), and
3. Large-diameter single-pedestal tanks of steel or concrete construction (load-bearing shear walls, as shown in Figure C15.7-3).

Unbraced multi-leg tanks are uncommon. These types of tanks differ in their behavior, redundancy, and resistance to overload. Multi-leg and small-diameter pedestal tanks have longer fundamental periods (typically greater than 2 seconds) than the shear wall type tanks (typically less than 2 seconds). The lateral load failure mechanisms usually are brace failure for multi-leg tanks, compression buckling for small-diameter steel tanks, compression or shear buckling for large-diameter steel tanks, and shear failure for large-diameter concrete tanks. Connection, welding, and reinforcement details require careful attention in order to mobilize the full strength of these structures. To provide a greater margin of safety, R factors used with elevated tanks typically are less than those for other comparable lateral load-resisting systems.

C15.7.10.4 Transfer of Lateral Forces into Support Tower. The vertical loads and shears transferred at the base of a tank or vessel supported by grillage or beams typically vary around the base due to the relative stiffness of the supports, settlements, and variations in construction. Such variations must be considered in the design for vertical and horizontal loads.

C15.7.10.5 Evaluation of Structures Sensitive to Buckling Failure. Nonbuilding structures with little structural redundancy for lateral loads may exhibit total failure when loaded only slightly beyond the design loads. Tanks and vessels supported on shell skirts or pedestals that are governed by buckling require evaluation for this critical condition.

The design spectral response acceleration, S_a, used in this evaluation includes site factors. The I/R coefficient is taken as 1.0 for this critical check. The structural capacity of the shell is taken as the critical buckling strength (that is, the factor of safety is 1.0). Vertical and orthogonal combinations need not be considered for this evaluation, since the probability of peak values occurring simultaneously is very low.

While the standard requires this evaluation only for structures assigned to Occupancy Category IV, it may be applied to any buckling-sensitive structure. Where such optional evaluations are performed, an R value of 2 or 3 can be used. In most cases, the design of the structure will be governed by this additional evaluation.

C15.7.10.7 Concrete Pedestal (Composite) Tanks. A composite elevated water-storage tank is comprised of a welded steel tank for watertight containment, a single pedestal concrete support structure, a foundation, and accessories. The lateral load-resisting system is a load-bearing concrete shear wall. Since the seismic provisions in ACI 371R-98 are based on an older edition of ASCE/SEI 7, appropriate bridging equations are provided in Section 15.7.10.7.

C15.7.11 Boilers and Pressure Vessels. The support system for boilers and pressure vessels must be designed for the seismic forces and displacements presented in the standard. Such design must include consideration of the support, the attachment of the support to the vessel (even if "integral"), and the body of the vessel itself, which is subject to local stresses imposed by the support connection.

C15.7.12 Liquid and Gas Spheres. The commentary in Section C15.7.11 also applies to liquid and gas spheres.

C15.7.13 Refrigerated Gas Liquid Storage Tanks and Vessels. Some refrigerated storage tanks and vessels, such as those storing LNG, are required to be designed for ground motions and performance goals in excess of those found in the standard, so such structures are outside the scope of the standard. All other welded steel refrigerated storage tanks and vessels must be designed in accordance with the requirements of the standard, the requirements of API 620, and the seismic requirements of API 650. Note that the seismic requirements of API 620 (10th Edition, Addendum 1) are note used as they are inconsistent with the requirements of the standard.

C15.7.14 Horizontal, Saddle Supported Vessels for Liquid or Vapor Storage. Past practice has been to assume that a horizontal, saddle supported vessel (including its contents) behaves as a rigid structure (with natural period, T, less than 0.06 seconds). For this situation, seismic forces would be determined using the requirements of Section 15.4.2. For large horizontal, saddle-supported vessels (length-to-diameter ratio of 6 or more), this assumption can be unconservative, so Section 15.7.14.3 requires that the natural period be determined assuming the vessel to be a simply supported beam.

Figure C15.7-1 Multi-leg braced steel tank.

Figure C15.7-2 Small-diameter single-pedestal steel tank.

(a) Steel **(b)** Concrete

Figure C15.7-3 Large-diameter single-pedestal tank.

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COMMENTARY TO CHAPTER 16, SEISMIC RESPONSE HISTORY PROCEDURES

C16.1 LINEAR RESPONSE HISTORY PROCEDURE

The standard does not require the use of linear response history analysis. However, the use of such analysis may be useful in validation of the results of the analysis methods presented in Chapter 12, or as a step in a series of analyses that culminate in a nonlinear response history analysis. While not commonly used in the past to design typical structures, this technique is seeing increased use in the design of some structures including structures that are neither damped nor base isolated.

The purpose of the linear response history procedure is to determine design forces for structural components and to compute displacements and story drifts, which must be within the limits specified by Table 12.12-1. In this sense, the linear response history procedure shares the force-based philosophy of the Equivalent Lateral Force (ELF) procedure and the Modal Response Spectrum (MRS) analysis procedure (both of which are specified in Chapter 12). Response history analysis offers several advantages over modal response spectrum analysis: it is more accurate mathematically, signs of response quantities (such as tension or compression in a brace) are not lost as a result of the combination of modal responses, and story drifts are computed more accurately. The principal disadvantages of response history analysis are the need to select and scale an appropriate suite of ground motions, and the necessity to perform analysis for several (usually seven) such motions. See Section C16.1.3 for discussion of ground motion selection and scaling techniques.

C16.1.1 Analysis Requirements. In response history analysis, the seismic hazard is characterized by a number of ground acceleration records. Using these records and a detailed mathematical model of the structure, nodal displacements and component forces are computed, step-by-step, by integration of the equations of motion. Two basic approaches for solving the equations may be used. In the first approach, called direct analysis, all the equilibrium equations for the entire system are solved simultaneously in each step. The number of equations solved equals the number of degrees of freedom in the structure.

In the second approach, called modal analysis, the equilibrium equations are transformed, by change of coordinates, into a number of single-degree-of-freedom (SDOF) systems. The maximum number of SDOF systems that can be formed is equal to the number of mass degrees of freedom in the structure. The SDOF equations are solved individually in time, and then the computed displacement histories are transformed back to the original coordinates and superimposed to obtain the system response history. The transformation of coordinates in the modal analysis approach is usually based on the undamped natural mode shapes of the structure. Other bases, such as a set of orthogonal load-dependent Ritz vectors, may be preferable in certain cases (Wilson et al., 1982).

Where modal analysis uses the full set of mode shapes and the damping ratios in each mode are identical to those obtained from the equations of motion used in the direct analysis, the two approaches produce identical results. A distinct advantage of the modal analysis approach is that a limited number of modes may be used to produce reasonably accurate results. While some accuracy is sacrificed where fewer modes are used, the computer resources required to perform the analysis are significantly less than those required for direct analysis. The number of modes required for a “reasonably” accurate analysis is discussed in Section C12.9.1.

C16.1.2 Modeling. The mathematical model used for linear response history analysis is usually identical to that used for modal response spectrum analysis, and it often reflects a preliminary design developed using the ELF procedure. The main modeling difference between response history analysis and modal response spectrum analysis is that the inherent damping (taken as 5 percent of critical) is included in the design response spectrum for response spectrum analysis, while it must be assigned explicitly for response history analysis.

In the modal analysis approach to response history analysis, damping is simply assigned to each mode that is included in the response (Wilson and Penzien, 1970). Although not specified in the standard, the damping used for each mode should be 5 percent of critical for consistency with the design response spectrum.

Direct response history analysis requires an explicit damping matrix. However, such a matrix cannot be formed from first principles; it is common to use a damping matrix that is proportional to the mass, the stiffness, or a linear combination of the two:

$$C = \alpha M + \beta K \qquad \text{(C16.1-1)}$$

where C is the damping matrix, M is the mass matrix, K is the stiffness matrix, and α and β are scalar constants of proportionality. Such damping is often referred to as Rayleigh damping.

The proportionality constants are determined as follows:

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = 2\begin{bmatrix} 1/\omega_a & \omega_a \\ 1/\omega_b & \omega_b \end{bmatrix}^{-1} \begin{Bmatrix} \xi_a \\ \xi_b \end{Bmatrix} \tag{C16.1-2}$$

where ξ_a and ξ_b are the desired damping ratios at any two system circular frequencies, ω_a and ω_b, where $\omega_b > \omega_a$. It is common, but not necessary, for the two specified frequencies to correspond to two of the system's lower natural frequencies (such as the first and third mode frequencies).

If both damping values are the same ($\xi = \xi_a = \xi_b$), which is usually the case, the mass and stiffness proportionality constants may be determined as follows:

$$\alpha = \xi \frac{2\omega_a \omega_b}{\omega_a + \omega_b} \qquad \beta = \xi \frac{2}{\omega_a + \omega_b} \tag{C16.1-3}$$

The advantage of Rayleigh damping is that it is simple to implement because all the analyst has to do is to specify the two proportionality constants α and β, and these can be established using Equation C16.1-2 given the two desired damping ratios and corresponding frequencies. The disadvantage is that the damping ratios increase with frequency and may cause the higher mode contributions to response to be over-damped. This effect is shown in Figure C16.1-1, where the damping ratios ξ have been set at 0.05 at frequencies of 4.2 and 12.5 radians per second. The damping at all other frequencies is given by the curve marked "Total". For frequencies above approximately 32 radians per second, the damping is greater than 10 percent of critical and may be excessive.

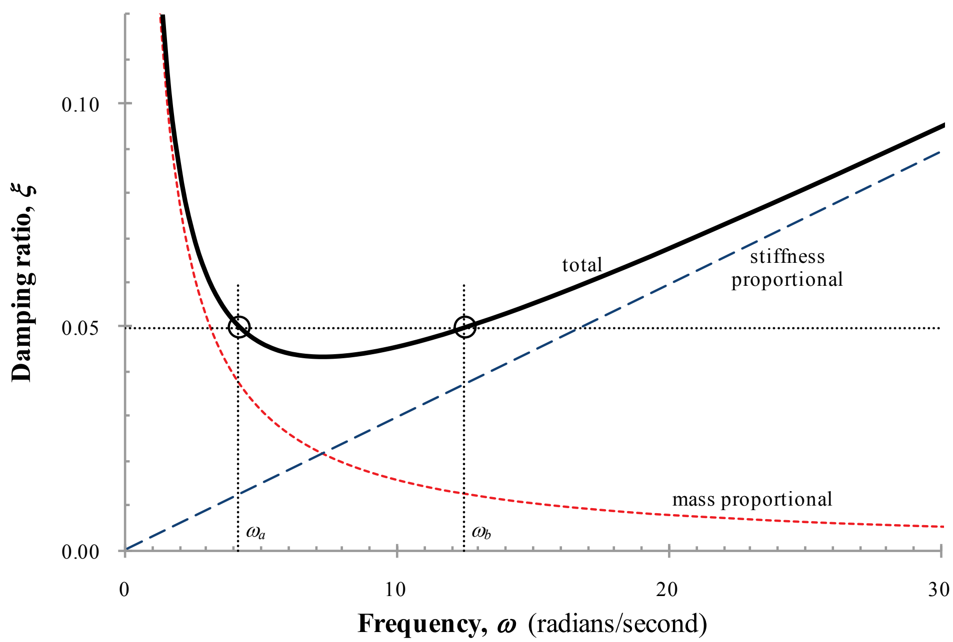

Figure C16.1-1 Example of Rayleigh damping.

C16.1.3 Ground Motion. One of the most demanding aspects of response history analysis is the selection and scaling of an appropriate suite of ground motions (Anderson and Bertero, 1987). It is considered appropriate to select records that have magnitudes, fault distances, source mechanisms, and soil conditions that are characteristic of the site. This poses quite a challenge even for sites in the western United States, where numerous records from large-magnitude earthquakes are available; it is virtually impossible in the central and eastern United States, where there are no recorded ground motions from large-magnitude events. (The web site for the Pacific Earthquake Engineering Research Center (PEER) provides a large number of ground motion acceleration records that may be used in response history analysis. In addition to the ground motions, the PEER site provides detailed background information on the source characteristics of the ground motions and on the instrument and site characteristics of the particular station that recorded the acceleration record.)

Because of the scarcity of available recorded motions, use of simulated ground motions is permitted. To this end, available records may be modified for site distance and soil conditions. Such modification is considered part of the ground motion selection.

The standard requires that at least three ground motions (or ground motion pairs, in the case of three-dimensional analysis) be used, and it provides an incentive for using at least seven motions (as discussed in Section C16.1.4).

The scaling technique specified in Sections 16.1.3.1 and 16.1.3.2 is one of several that have been proposed. See Shome and Cornell (1998), Shome et al. (1998), Somerville et al. (1998), Mehrain and Naiem (2003), and Iervolino and Cornell (2005) for background on ground motion selection and scaling. (Applied Technology Council (ATC) Projects 58 and 63 are also investigating scaling techniques.)

C16.1.3.1 Two-Dimensional Analysis. This scaling method begins with ground motions that have been selected (and modified as necessary) to have magnitude, distance, and site conditions compatible with the maximum considered earthquake. The 5 percent damped pseudo-acceleration response spectra for these records are scaled for consistency with the design ground motion spectrum shown in Figure 11.4-1. For two-dimensional analysis, the ground motion spectra must be scaled such that the average of the spectra is not less than the design spectrum in the period range from $0.2T$ to $1.5T$, where T is the fundamental period of vibration of the structure being designed. The short period of the range ($0.2T$) is set to capture higher mode response, and the long period of the range ($1.5T$) is set to allow for period lengthening that would be associated with inelastic response.

C16.1.3.2 Three-Dimensional Analysis. Approaches to scaling ground motions for three-dimensional analysis are similar to those for two-dimensional analysis. The two orthogonal components within each pair must have the same scale factor, but the individual pairs may have different scale factors. Within 10 kilometers of a fault, ground motion components often are selected to represent fault-normal and fault-parallel directions, but this is not required. For certain structures, the response under both horizontal and vertical ground motions should be considered. It is noted, however, that vertical ground motion spectra are not readily available, so the scaling of the vertical components of ground motion would be problematic.

The 1.3 factor in the comparison of the average of the SRSS spectra to the design spectrum is intended to compensate for the increase associated with taking the SRSS of the two components of each ground motion pair. If the two components are perfectly correlated (identical response spectra in both directions), the SRSS would be larger than the average by the square root of 2. Because real ground motions are not perfectly correlated, a smaller factor is acceptable. The judgment of the writers (after two cycles of revision) is to allow a factor of 1.3 and to allow the results to be low by as much as 10 percent (producing an effective factor of 1.18).

Given a set of appropriate ground motions, there are an infinite number of scaling factors that may be applied to the individual motions to meet the requirements of Sections 16.1.3.1 and 16.1.3.2. Thus, two analysts, working with the same set of ground motions, are likely to produce a different set of scale factors. While this difference in scaling would have little impact in linear analysis, it may lead to vastly different results in nonlinear analysis. For this reason, the process of selection and scaling of ground motions should be included in the design review (Section 16.2.5 of the standard) that is required wherever nonlinear response history analysis is used.

Both amplitude scaling and spectral matching procedures can be used to satisfy the scaling technique specified in Sections 16.1.3.1 and16.1.3.2. Both procedures provide reasonable estimates of mean response for the individual response parameters. Spectral matching can provide mean estimates with a smaller suite of motions, although seven suites are still required as outlined in Section 16.1.4. Neither scaling approach, however, is adequate to give an accurate estimate of the variability, although amplitude scaling gives a better understanding of the potential variability than spectral matching.

C16.1.4 Response Parameters. The responses derived from the response history analysis are multiplied by I to provide enhanced strength and stiffness for more important facilities, and are divided by R to account for inelastic behavior. For consistency with the ELF procedure and the MRS analysis procedure, the displacements computed from the response histories that have been further modified by I/R should be multiplied by C_d to obtain the displacement histories to use for computing the story drift histories. (The requirement to multiply displacements by C_d was incorrectly omitted in the standard.)

If for any ground motion the peak base shear is less than that computed from Equation 12.8-5 or 12.8-6, the entire response history must be scaled up such that the peak base shear is not less than that computed from Equation 12.8-5 or 12.8-6, as applicable. The base shear typically is computed from component elastic forces. A slightly different shear would be computed from the total inertial forces, with the difference being due to damping. Note that while the results of MRS analysis must be scaled up such that the corresponding base shear is not less than 85 percent of the base shear that would be computed from an ELF analysis (see Section C12.9.4), the scaling for linear response history analysis considers only the applicable minimum base shear coefficient.

If seven or more ground motions are used, the design values may be taken as the average of the scaled values from the response history analysis. This provides some difficulty for components for which the capacity depends on multiple values. For a column, for example, both the axial force and the concurrent bending moment are needed to compare demand and capacity. In that instance, if seven or more ground motions are used, the column is deemed suitable if the average of the seven peak demand-to-capacity ratios for the column is less than 1.0. Where fewer than seven ground motions are used, the column is deemed suitable if the maximum demand-to-capacity ratio is less than 1.0.

The direction of loading requirements of Section 12.5 and the modeling requirements of Section 12.7 apply to response history analysis, but Chapter 16 of the standard does not address additional requirements such as accidental torsion, amplification of accidental torsion, or detailed consideration of P-delta effects. These effects should be included in a manner consistent with the requirements of Section 12.9.

C16.2 NONLINEAR RESPONSE HISTORY PROCEDURE

Nonlinear response history analysis is not used as part of the normal design process for typical structures. In some cases, however, nonlinear analysis is recommended, and in certain cases required, to obtain a more realistic assessment of structural response and verify the results of simpler methods of analysis. Such is the case for systems with friction-based passive energy dissipation devices, nonlinear viscous dampers, seismically isolated systems, self-centering systems, or systems that have components with highly irregular force-deformation relationships.

The principal aim of nonlinear response history analysis is to determine if the computed deformations of the structure are within appropriate limits. Strength requirements for the designated lateral load-resisting elements do not apply because element strengths are established prior to the analysis. These initial strengths typically are determined from a preliminary design using linear analysis.

The nonlinear response history analysis may also provide useful information on the strength requirements for nonstructural components, which are often assumed to remain elastic in the analysis.

Where displacements computed from the nonlinear response history analysis are excessive, a typical remedy is to increase the stiffness of the structure, which is likely to affect the computed strength.

Nonlinear response history analysis offers several advantages over linear response history analysis, including the ability to model a wide variety of nonlinear material behaviors, geometric nonlinearities (including large displacement effects), gap opening and contact behavior, and nonclassical damping, and to identify the likely spatial and temporal distributions of inelasticity. Nonlinear response history analysis has several disadvantages, including increased effort to develop the analytical model, increased time to perform the analysis (which is often complicated by difficulties in obtaining converged solutions), sensitivity of computed response to system parameters, and the inapplicability of superposition to combine live, dead, and seismic load effects.

C16.2.1 Analysis Requirements. Nonlinear response history analysis of structures with widely distributed inelastic behavior is usually carried out using the direct analysis approach (described in Section C16.1.1), wherein all equations are solved simultaneously at each time step. In some cases, it is possible to use a highly efficient nonlinear modal analysis approach called Fast Nonlinear Analysis, or FNA (Wilson, 2004). The class of nonlinear structures that may be analyzed by the FNA approach consists of structures with a very limited number of discrete sources of well-defined nonlinear behavior. Such structures include seismically isolated structures and structures with damping systems. Because of the limited applicability of FNA, this commentary discusses only the direct analysis approach.

The sensitivity of nonlinear response history analysis may be evidenced by results that appear to be chaotic or even counter-intuitive, although they may be correct. For example, it is possible for the analysis to predict that a structure collapses when subjected to a given ground motion, while surviving at higher intensity of the same motion. Similarly, the results from analyses of the same structure for several ground motions with similar spectral shapes and amplitudes often differ substantially. A systematic approach to assess the sensitivity of structures to different ground motions and structural system parameters, using Incremental Dynamic Analysis (IDA), is reported by Vamvatsikos (2002). The IDA method has become an important tool in earthquake engineering research.

C16.2.2 Modeling. Nonlinear response history analysis requires a mathematical description of the hysteretic behavior of those portions of the structure that are expected to exhibit inelastic behavior during an earthquake. Such models must reflect the expected properties, accounting for the following effects as appropriate:

1. Material overstrength and strain hardening
2. Cyclic degradation of stiffness and strength
3. In-cycle degradation of stiffness and strength (Applied Technology Council, 2009)
4. Pinching
5. Buckling
6. Axial-flexural-shear interaction

Most of the available mathematical models are phenomenological and represent yielding portions as distinct elements (such as plastic hinges). More exact analysis may be performed by subdividing yielding portions into a number of slices or fibers. This more exact approach is preferable but is more computationally demanding.

An inelastic three-dimensional analysis is particularly useful for buildings that are prone to torsional response in plan, even where the main seismic force-resisting systems resist loads predominantly in their own plane. If only two-dimensional software is available, a “pseudo” three-dimensional analysis may be performed (Mehrain and Naeim, 2003).

In moment resisting steel frames, the elastic and inelastic behavior of beam-column joint regions should be modeled explicitly. P-delta effects should be considered explicitly in the analysis. Nonstructural components also should be included in the model if it is expected that their stiffness and strength have a significant effect on the response.

Nonlinear response history analysis requires that inherent damping be set for the structure. As for linear response history analysis, nonlinear response history analysis typically is performed assuming inherent damping of 5 percent of critical. Some analysts and designers advocate the use of lower levels of inherent damping (perhaps 2 percent of critical), especially for steel frames, but there is no widespread agreement on this point.

The mechanism used to include inherent damping in the analytical model is critically important to the accuracy of the computed response. Most nonlinear analysis programs use a form of Rayleigh damping, wherein the damping matrix (used for direct integration of the equations of motion) is represented as a linear combination of the mass and stiffness matrices. (See Section C16.1.2.) If the damping matrix is based on the initial stiffness of the system, artificial damping may be generated by system yielding. In some cases, the artificial damping can completely skew the computed response (Chrisp, 1980; Carr, 2004; Charney, 2006; Hall, 2006). One method to counter this occurrence is to base the damping matrix on the mass and the instantaneous tangent stiffness. (Where basing the damping on tangent stiffness, care must be taken so that the damping is not negative when the tangent stiffness is negative.) Other approaches have been suggested, such as capped Rayleigh damping (Hall, 2006) and hysteretic damping (Charney, 2006). Several commercial programs, including SAP2000, Perform 2D, and Ruaomoko, provide for tangent stiffness-based damping.

Three-dimensional analysis must be used where certain plan irregularities are present. For structures composed of two-dimensional seismic force-resisting elements connected by floor and roof diaphragms, the diaphragms should be modeled as flexible in-plane, particularly where the vertical elements of the seismic force-resisting system are of different types (such as moment frames and walls). Where structures are modeled in three dimensions, axial force-biaxial bending interaction should be considered for corner columns, nonrectangular walls, and other similar elements.

As mentioned above, P-delta effects should be included where significant. The significance of P-delta effects on the overall response may be assessed by performing analyses with and without P-delta effects, and comparing story drift response histories. Destabilizing effects of gravity loads are often manifested by accumulated residual deformations, and these deformations, if not controlled, can lead to dynamic instability of the structure.

C16.2.3 Ground Motion and Other Loading. Since linear superposition cannot be used with nonlinear analysis, each response history analysis must begin with an initial gravity load, consisting of the expected dead load and live load. The live load may be as little as 25 percent of the unfactored design live load because multiple transient loads are unlikely to attain their maxima simultaneously.

C16.2.4 Response Parameters. As discussed above, the principal aim of nonlinear response history analysis is to determine deformation demands in structural and nonstructural components for comparison with accepted limits. Where at least seven ground motions are used, the member and connection deformations may be taken as the average of the values computed from the analyses. If fewer than seven motions are used, the maximum values among all analyses must be used. It is very important to note, however, that assessment of deformations in this manner should not be done without careful inspection of the story displacement histories of each analysis. It is possible that the maximum displacement or drift may be completely dominated by the response from one ground motion, and such dominance, when due to ratcheting (increasing deformations in one direction resulting in a high residual deformation), may be a sign of imminent dynamic instability. Where these kinds of dynamic instabilities are present, the analyst should attempt to determine the system characteristics that produce such effects. The ground motion that produces dynamic instability should not be replaced with one that does not.

C16.2.4.1 Member Strength. The strength design load combinations of Section 12.4 need not be assessed because linear combinations of load are not applicable in nonlinear analysis. Overstrength effects are evaluated directly since hysteretic force-deformation relationships are modeled explicitly and the material properties so used include overstrength and strain hardening (as required by Section 16.2.2).

C16.2.4.2 Member Deformation. This section requires that member and connection deformations be assessed on the basis of tests performed for similar configurations.

C16.2.4.3 Story Drift. The 25 percent increase in allowable story drift is provided because the nonlinear analysis is generally more accurate than linear analysis and because member deformations are assessed explicitly.

C16.2.5 Design Review. As discussed above, nonlinear response history analysis is quite complex, and the results may be strongly influenced by subtle changes in ground motion or system properties. Hence, such analysis must only be conducted by experienced professionals with training in engineering seismology, earthquake engineering, structural dynamics, stability, nonlinear analysis, and inelastic behavior of structures. Regardless of the level of expertise of the individual or individuals who perform the analysis and design, a design (peer) review of the structural system and the nonlinear analysis is required wherever the design is based on the nonlinear response history procedure.

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COMMENTARY TO CHAPTER 17, SEISMIC DESIGN REQUIREMENTS FOR SEISMICALLY ISOLATED STRUCTURES

C17.1 GENERAL

Seismic isolation, commonly referred to as base isolation, is a method used to decouple substantially the response of a structure from potentially damaging earthquake motions. This decoupling can result in response that is reduced significantly from that of a conventional, fixed-base building.

The potential advantages of seismic isolation and the recent advancements in isolation-system technology have led to the design and construction of a large number of seismically isolated buildings and bridges in the United States.

Design requirements for seismically isolated structures were first codified in the United States as an appendix to the 1991 *Uniform Building Code*, based on “General Requirements for the Design and Construction of Seismic-Isolated Structures” developed by the Structural Engineers Association of California State Seismology Committee. In the intervening years, those provisions have developed along two parallel tracks into the design requirements in Chapter 17 of the standard and the rehabilitation requirements in Section 9.2 of ASCE/SEI 41, *Seismic Rehabilitation of Existing Buildings*. The design and analysis methods of both standards are quite similar, but ASCE/SEI 41 permits more liberal design for the superstructure of rehabilitated buildings. The AASHTO *Guide Specification for Seismic Isolation Design* provides a systematic approach to determining bounding values of mechanical properties of isolators for analysis and design. Rather than addressing a specific method of seismic isolation, the standard provides general design requirements applicable to a wide range of possible seismic isolation systems. Because the design requirements are general, testing of isolation-system hardware is required to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Use of isolation systems whose adequacy is not proved by testing is prohibited. In general, acceptable systems: (a) remain stable when subjected to design displacements, (b) provide increasing resistance with increasing displacement, (c) do not degrade under repeated cyclic load, and (d) have quantifiable engineering parameters (such as force-deflection characteristics and damping).

The force-deflection behavior of isolation systems falls into four categories, as shown in Figure C17.1-1, where each idealized curve has the same design displacement, D_D. A linear isolation system (Curve A) has an effective period independent of displacement, and the force generated in the superstructure is directly proportional to the displacement of the isolation system.

with increasing displacement, the procedures of the standard cannot be applied, and use of the system is prohibited.

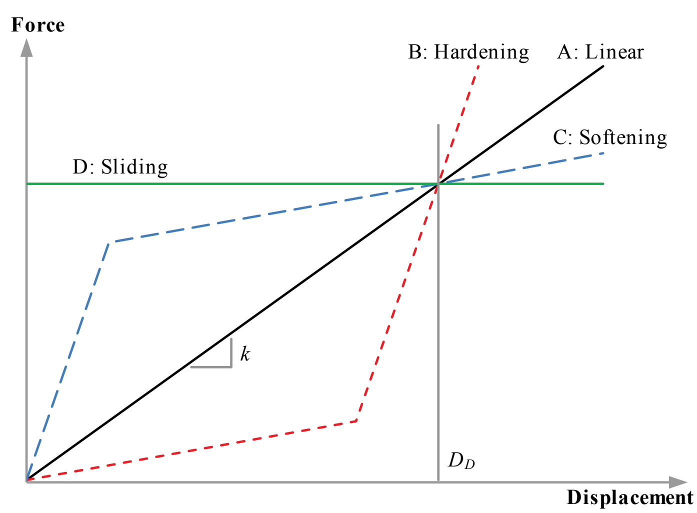

Figure C17.1-1 Idealized force-deflection relationships for isolation systems (stiffness effects of sacrificial wind-restraint systems not shown for clarity).

A hardening isolation system (Curve B) is soft initially (long effective period) and then stiffens (effective period shortens) as displacement increases. Where displacements exceed the design displacement, the superstructure is subjected to larger forces and the isolation system to smaller displacements than for a comparable linear system.

A softening isolation system (Curve C) is stiff initially (short effective period) and then softens (effective period lengthens) as displacement increases. Where displacements exceed the design displacement, the superstructure is subjected to smaller forces and the isolation system to larger displacements than for a comparable linear system.

The response of a purely sliding isolation system (Curve D) is governed by the friction force at the sliding interface. For increasing displacement, the effective period lengthens, and loads on the superstructure remain constant. For isolation systems governed solely by friction forces, the total displacement due to repeated earthquake cycles is highly dependent on the characteristics of the ground motion and may exceed the design displacement, D_D. Since such systems do not have increasing resistance

Chapter 17 provides isolator design displacements, shear forces for structural design, and other specific requirements for seismically isolated structures. All other design requirements, including loads (other than seismic), load combinations, allowable forces and stresses, and horizontal shear distribution, are the same as those for conventional, fixed-base structures.

C17.1.1 Variations in Material Properties. For analysis, the mechanical properties of seismic isolators generally are based on values provided by isolator manufacturers. Values of the mechanical properties should be in the range that accounts for natural variability and uncertainty, and variability of properties among isolater of different manufacturers. Examples may be found in Constantinou et al. (2007b). Prototype testing is used to confirm the values assumed for design. Unlike conventional materials whose properties do not vary substantially with time, the materials used in seismic isolators have properties that generally will vary with time. Because mechanical properties can vary over the life of a structure and the testing protocol of Section 17.8 cannot account for the effects of aging, contamination, scragging (temporary degradation of mechanical properties with repeated cycling), temperature, velocity effects, and wear, the designer must account for these effects by explicit analysis. One approach to accommodate these effects, introduced in Constantinou et al. (1999), is to use property modification factors. Information on variations in material properties of seismic isolators and dampers is reported in Constantinou et al. (2007).

C17.2 GENERAL DESIGN REQUIREMENTS

Ideally, most of the lateral displacement of an isolated structure will be accommodated by deformation of the isolation system rather than distortion of the structure above. Accordingly, the seismic-force-resisting system of the structure above the isolation system is designed to have sufficient stiffness and strength to avoid large, inelastic displacements. Therefore, the standard contains criteria that limit the inelastic response of the structure above the isolation system. Although damage control is not an explicit objective of the standard, design to limit inelastic response of the structural system also will reduce the level of damage that would otherwise occur during an earthquake. In general, isolated structures designed in accordance with the standard are expected:

1. To resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural components, or building contents and
2. To resist major levels of earthquake ground motion without failure of the isolation system, significant damage to structural elements, extensive damage to nonstructural components, or major disruption to facility function.

Isolated structures are expected to perform much better than fixed-base structures during moderate and major earthquakes. Table C17.2-1 compares the performance expected for isolated and fixed-base structures designed in accordance with the standard.

Table C17.2-1 Performance Expected for Minor, Moderate, and Major Earthquakes[a]

Performance Measure	Earthquake Ground Motion Level		
	Minor	Moderate	Major
Life safety: loss of life or serious injury is not expected	F, I	F, I	F, I
Structural damage: significant structural damage is not expected	F, I	F, I	I
Nonstructural damage: significant nonstructural or contents damage is not expected	F, I	I	I

[a] F indicates fixed base; I indicates isolated.

Loss of function is not included in Table C17.2-1. For certain fixed-base facilities, loss of function would not be expected unless there is significant structural damage causing closure or restricted access to the building. In other cases, a facility with only limited or no structural damage would not be functional as a result of damage to vital nonstructural components or contents. Isolation would be expected to mitigate structural and nonstructural damage and to protect the facility against loss of function.

C17.2.4 Isolation System

C17.2.4.1 Environmental Conditions. Environmental conditions that may adversely affect isolation system performance must be investigated thoroughly. Related research has been conducted since the 1970s in Europe, Japan, New Zealand, and the United States.

C17.2.4.2 Wind Forces. Lateral displacement over the depth of the isolator zone resulting from wind loads must be limited to a value similar to that required for other story heights.

C17.2.4.3 Fire Resistance. While fire may adversely affect the lateral performance of the isolation system, its gravity-load resistance must be maintained as required for other elements of the structure.

C17.2.4.4 Lateral Restoring Force. The restoring-force requirement is intended to limit residual displacement as a result of an earthquake, so that the isolated structure will survive aftershocks and future earthquakes.

C17.2.4.5 Displacement Restraint. The use of a displacement restraint is discouraged. Where a displacement restraint system is used, explicit analysis of the isolated structure for maximum considered earthquake response is required to account for the effects of engaging the displacement restraint.

C17.2.4.6 Vertical-load Stability. The vertical loads to be used in checking the stability of any given isolator should be calculated using bounding values of dead load and live load and the peak earthquake demand of the maximum considered earthquake. Since earthquake loads are reversible in nature, peak earthquake load should be combined with bounding values of dead and live load in a manner which produces both the maximum downward force and the maximum upward force on any isolator. Stability of each isolator should be verified for these two extreme values of vertical load at peak maximum considered earthquake displacement of the isolation system.

C17.2.4.7 Overturning. The intent of this requirement is to prevent both global structural overturning and overstress of elements due to local uplift. Isolator uplift is acceptable so long as the isolation system does not disengage from its horizontal-resisting connection detail. The connection details used in some isolation systems are such that tension is not permitted on the system. Where the tension capacity of an isolator is used to resist uplift forces, design and testing in accordance with Sections 17.2.4.6 and 17.8.2.5 must be performed to demonstrate the adequacy of the system to resist tension forces at the total maximum displacement.

C17.2.4.8 Inspection and Replacement. Although most isolation systems will not need to be replaced after an earthquake, access for inspection and replacement must be provided, and periodic inspection is required. After an earthquake, the isolation system should be inspected and any damaged elements replaced or repaired.

C17.2.4.9 Quality Control. A testing and inspection program is necessary for both fabrication and installation of the isolator units. Because seismic isolation is a rapidly evolving technology, it may be difficult to reference standards for testing and inspection. Reference can be made to standards for some materials, such as elastomeric bearings (ASTM D 4014). Similar standards are yet to be developed for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality must be developed for each project. The requirements will vary depending on the type of isolation system used.

C17.2.5 Structural System

C17.2.5.2 Building Separations. A minimum separation between the isolated structure and rigid obstructions is required to allow free movement of the superstructure in all lateral directions during an earthquake.

C17.2.6 Elements of Structures and Nonstructural Components. To accommodate the differential movement between the isolated building and the ground, flexible utility connections are required. In addition, stiff elements crossing the isolation interface (such as stairs, elevator shafts, and walls) must be detailed to accommodate the total maximum displacement without compromising life safety.

C17.3 GROUND MOTION FOR ISOLATED STRUCTURES

C17.3.1 Design Spectra. Seismically isolated structures located on Site Class F sites and on sites with $S_1 \geq 0.6$ must be analyzed using response history analysis. For those cases, the response spectra must be site-specific in order to account, in the analysis, for near-fault effects and for soft soil conditions, both of which are known to be important in the assessment of displacement demands in seismically isolated structures.

C17.3.2 Ground Motion Histories. The selection and scaling of ground motions for response history analysis requires fitting to the response spectra in the period range of $0.5T_D$ to $1.25T_M$, a range that is different from that for conventional structures $0.2T$ to $1.5T$. The following sections provide background on the two period ranges:

1. Period Range – Isolated Structures. The effective (fundamental) period of an isolated structure is based on amplitude-dependent, nonlinear (pushover) stiffness properties of the isolation system. The effective periods, T_D and T_M, correspond to design earthquake displacement and maximum considered earthquake displacement, respectively, in the direction under consideration. Values of effective (fundamental) periods, T_D and T_M, are typically in the range of 2 to 4 seconds, and the value of the effective period, T_D, typically is 15 to 25 percent less than the corresponding value of effective period, T_M.

   The response of an isolated structure is dominated by the fundamental mode in the direction of interest. The specified period range, $0.5T_D$ to $1.25T_M$, conservatively bounds amplitude-dependent values of the effective (fundamental) period of the isolated structure in the direction of interest, considering that individual earthquake records can affect response at effective periods somewhat longer than T_M, or significantly shorter than T_D.

2. Period Range – Conventional, Fixed-Base Structures. The fundamental period, T, of a conventional, fixed-base structure is based on amplitude-independent, linear-elastic stiffness properties of the structure. In general, response of conventional, fixed-base structures is influenced by both the fundamental mode and higher modes in the direction under consideration. The period range, $0.2T$ to $1.5T$, is intended to bound the fundamental period, considering some period lengthening due to nonlinear response of the structure (that is, inelastic periods up to $1.5T$) and periods corresponding to the more significant higher modes (that is, second and possibly third modes in the direction of interest).

C17.4 ANALYSIS PROCEDURE SELECTION

Three different analysis procedures are available for determining design-level seismic loads: the equivalent lateral force procedure, the response spectrum procedure, and the response history procedure. For the first procedure simple, lateral-force formulas (similar to those for conventional, fixed-base structures) are used to determine peak lateral displacement and design forces as a function of spectral acceleration and isolated-structure period and damping. For the second and third procedures, which are required for geometrically complex or especially flexible buildings, dynamic analysis (either the response spectrum procedure or the response history procedure) is used to determine peak response of the isolated structure.

The three procedures are based on the same level of seismic input and require a similar level of performance from the building. Where more complex analysis procedures are used, slightly lower design forces and displacements are permitted. The design requirements for the structural system are based on the design earthquake, taken as two-thirds of the maximum considered earthquake. The isolation system—including all connections, supporting structural elements, and the "gap"—is required to be designed (and tested) for 100 percent of maximum considered earthquake demand. Structural elements above the isolation system are not required to be designed for the full effects of the design earthquake but may be designed for slightly reduced loads (that is, loads reduced by a factor of up to 2) if the structural system is able to respond inelastically without sustaining significant damage. A similar fixed-base structure would be designed for loads reduced by a factor of 8 rather than 2.

This section delineates the requirements for the use of the equivalent lateral force procedure and dynamic methods of analysis. The limitations on the simplified lateral-force design procedure are quite restrictive. Limitations relate to the site location with respect to major, active faults; soil conditions of the site; the height, regularity, and stiffness characteristics of the building; and selected characteristics of the isolation system. Response-history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. Isolated structures with a "nonlinear" isolation system including, but not limited to, isolation systems with effective damping values greater than 30 percent of critical, isolation systems incapable of producing a significant restoring force, and isolation systems that restrain or limit extreme earthquake displacement; and
2. Isolated structures located on a Class E or Class F site (that is, a soft soil site that amplifies long-period ground motions).

Lower-bound limits on isolation system design displacements and structural-design forces are specified by the standard in Section 17.6 as a percentage of the values prescribed by the equivalent lateral force procedure, even where dynamic analysis is used as the basis for design. These lower-bound limits on key design parameters provide consistency in the design of isolated structures and serve as a "safety net" against gross underdesign. Table C17.4-1 provides a summary of the lower-bound limits on dynamic analysis specified by the standard.

C17.5 EQUIVALENT LATERAL FORCE PROCEDURE

C17.5.3 Minimum Lateral Displacements. The lateral displacement given by Equation 17.5-1 approximates peak design earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_D, and equivalent viscous damping, β_D. Similarly, the lateral displacement given by Equation 17.5-3 approximates peak maximum considered earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_M, and equivalent viscous damping, β_M.

Table C17.4-1 Lower-Bound Limits on Dynamic Procedures Specified in Relation to ELF Procedure Requirements

Design Parameter	ELF Procedure	Dynamic Procedure	
		Response Spectrum	Response History
Design displacement – D_D	$D_D = (g/4\pi^2)(S_{D1}T_D/B_D)$	—	—
Total design displacement - D_{TD}	$D_{TD} \geq 1.1D_D$	$\geq 0.9D_{TD}$	$\geq 0.9D_{TD}$
Maximum displacement – D_M	$D_M = (g/4\pi^2)(S_{M1}T_M/B_M)$	—	—
Total maximum displacement - D_{TM}	$D_{TM} \geq 1.1D_M$	$\geq 0.8D_{TM}$	$\geq 0.8D_{TM}$
Design shear – V_b (at or below the isolation system)	$V_b = k_{Dmax}D_D$	$\geq 0.9V_b$	$\geq 0.9V_b$
Design shear – V_s ("regular" superstructure)	$V_s = k_{Dmax}D_D/R_I$	$\geq 0.8V_s$	$\geq 0.6V_s$
Design shear – V_s ("irregular" superstructure)	$V_s = k_{Dmax}D_D/R_I$	$\geq 1.0V_s$	$\geq 0.8V_s$
Drift (calculated using R_I for C_d)	$0.015h_{sx}$	$0.015h_{sx}$	$0.020h_{sx}$

Equation 17.5-1 is an estimate of peak displacement in the isolation system for the design earthquake. In this equation, the spectral acceleration term, S_{D1}, is the same as that required for design of a conventional, fixed-base structure of period, T_D. A damping term, B_D, is used to decrease (or increase) the computed displacement where the equivalent damping coefficient of the isolation system is greater (or smaller) than 5 percent of critical damping. Values of coefficient B_D (or B_M for the maximum considered earthquake) are given in Table 17.5-1 for different values of isolation system damping, β_D (or β_M).

A comparison of values obtained from Equation 17.5-1 and those obtained from nonlinear time-history analyses are given in Kircher et al. (1988) and Constantinou et al. (1993).

Consideration should be given to possible differences in the properties used for design of the isolation system and those of the isolation system as installed in the structure. Similarly, consideration should be given to possible changes in isolation system properties due to different design conditions or load combinations. If the true deformational characteristics of the isolation system are not stable or if they vary with the nature of the load (being rate-, amplitude-, or time-dependent), the design displacements should be based on deformational characteristics of the isolation system that give the largest possible deflection (k_{Dmin}), the design forces should be based on deformational characteristics of the isolation system that give the largest possible force (k_{Dmax}), and the damping level used to determine design displacements and forces should be based on deformational characteristics of the isolation system that represent the minimum amount of energy dissipated during cyclic response at the design level.

The isolation system for a seismically isolated structure should be configured to minimize eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system. In this way, the effect of torsion on the displacement of isolation elements will be reduced. As for conventional structures, allowance must be made for accidental eccentricity in both horizontal directions. Figure C17.5-1 illustrates the terminology used in the standard. Equation 17.5-5 (or Equation 17.5-6 for the maximum considered earthquake) provides a simplified formula for estimating the response due to torsion in lieu of a more refined analysis. The additional component of displacement due to torsion increases the design displacement at the corner of a structure by about 15 percent (for one perfectly square in plan) to about 30 percent (for one very long and rectangular in plan) if the eccentricity is 5 percent of the maximum plan dimension. These calculated torsional displacements are for structures with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the structure, or certain sliding systems that minimize the effects of mass eccentricity, will have smaller torsional displacements. The standard permits values of D_{TD} as small as $1.1D_D$, with proper justification.

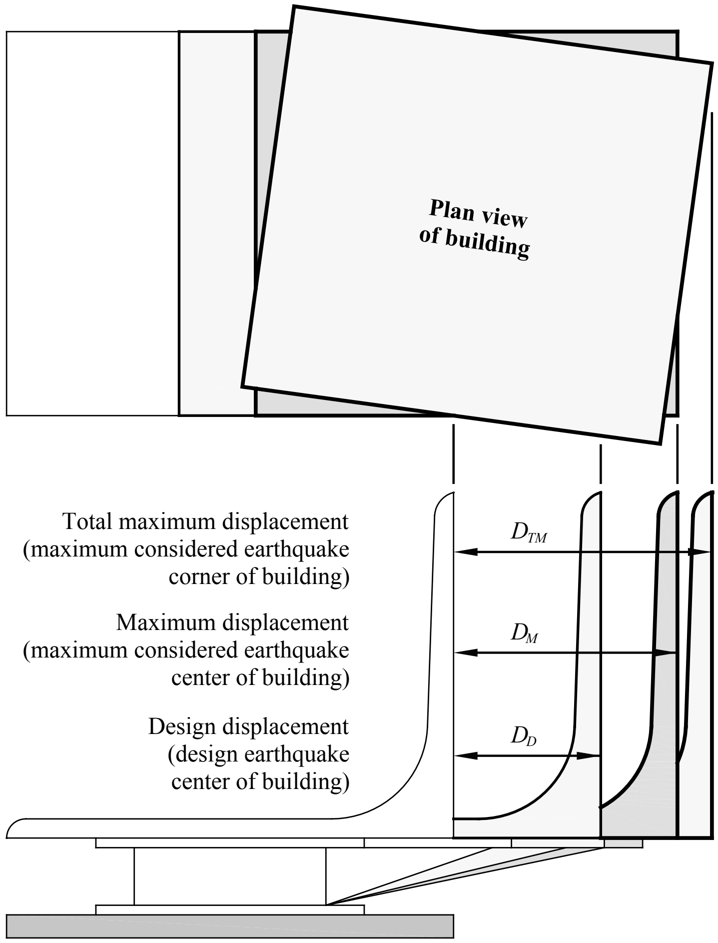

Figure C17.5-1 Displacement terminology.

C17.5.4 Minimum Lateral Forces. Figure C17.5-2 illustrates the terminology for elements at, below, and above the isolation system. Equation 17.5-7 specifies the peak seismic shear for design of all structural elements at or below the isolation system (without reduction for ductile response). Equation 17.5-8 specifies the peak seismic shear for design of structural elements above the isolation system. For structures that have appreciable inelastic-deformation capability, this equation includes an effective reduction factor of up to 2 for response beyond the strength-design level.

The reduction factor is based on use of strength-design procedures. A factor of at least 2 is assumed to exist between the design-force level and the true-yield level of the structural system. An investigation of 10 specific buildings indicated that this factor varied between 2 and 5 (ATC, 1982). Thus, a reduction factor of 2 is appropriate to produce a structural system that remains essentially elastic for the design earthquake.

In Section 17.5.4.3, the limits given on V_S provide at least a factor of 1.5 between the nominal yield level of the superstructure and (a) the yield level of the isolation system, (b) the ultimate capacity of a sacrificial wind-restraint system that is intended to fail and release the superstructure during significant lateral load, or (c) the break-away friction level of a sliding system.

These limits are needed so that the superstructure will not yield prematurely before the isolation system has been activated and significantly displaced.

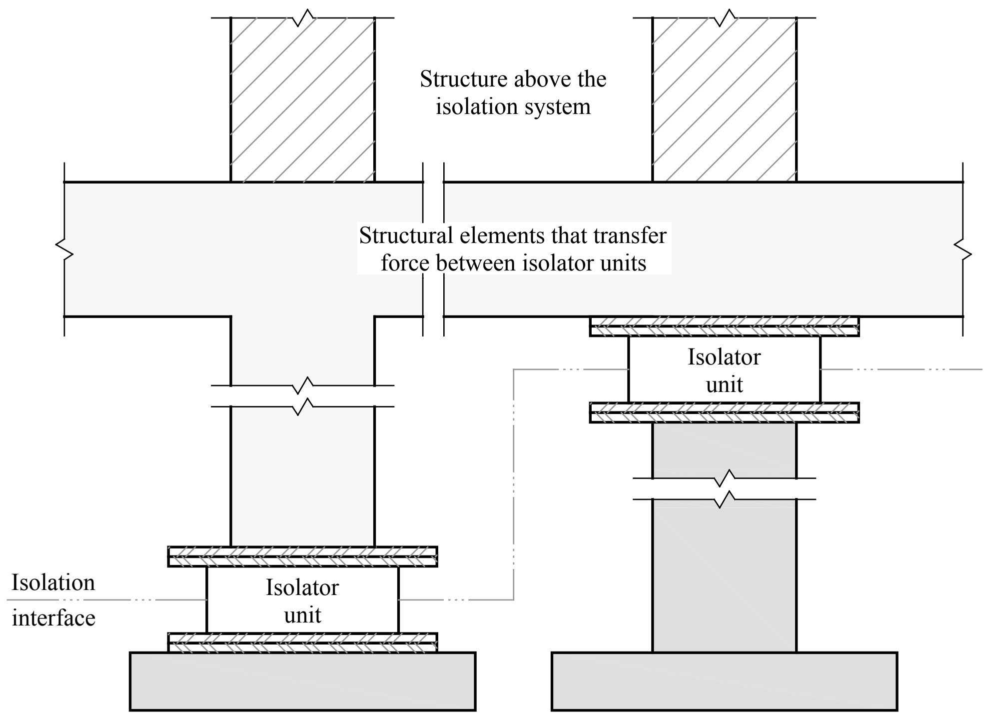

Figure C17.5-2 Isolation system terminology.

The design shear force, V_S, specified in this section results in an isolated structural system being subjected to significantly lower inelastic demands than a conventionally designed structural system. Further reduction in V_S, such that the inelastic demand on a seismically isolated structure would be the same as the inelastic demand on a conventionally designed structure, was not considered during development of these requirements but may be considered in the future.

Using a smaller value of R_I in Equation 17.5-8 will reduce or eliminate inelastic response of the superstructure for the design-basis event, thus further improving the structural performance.

C17.5.5 Vertical Distribution of Forces. Equation 17.5-9 produces a vertical distribution of lateral forces consistent with a triangular profile of seismic acceleration over the height of the structure above the isolation interface. Kircher et al. (1988) and Constantinou et al. (1993) show that Equation 17.5-9 provides a conservative estimate of the distributions obtained from more detailed, nonlinear analysis studies for the type of structures for which use of Equation 17.5-9 is allowed.

C17.5.6 Drift Limits. The maximum story drift permitted for design of isolated structures is constant for all Occupancy Categories, as shown in Table C17.5-1. For comparison, the drift limits prescribed by the standard for fixed-base structures also are summarized in that table.

Table C17.5-1 Comparison of Drift Limits for Fixed-Base and Isolated Structures

Structure	Occupancy Category	Fixed-Base	Isolated
Buildings (other than masonry) four stories or less in height with component drift design	I or II	$0.025h_{sx}/(C_d/R)$	$0.015h_{sx}$
	III	$0.020h_{sx}/(C_d/R)$	$0.015h_{sx}$
	IV	$0.015h_{sx}/(C_d/R)$	$0.015h_{sx}$
Other (non-masonry) buildings	I or II	$0.020h_{sx}/(C_d/R)$	$0.015h_{sx}$
	III	$0.015h_{sx}/(C_d/R)$	$0.015h_{sx}$
	IV	$0.010h_{sx}/(C_d/R)$	$0.015h_{sx}$

Drift limits in Table C17.5-1 are divided by C_d/R for fixed-base structures since displacements calculated for lateral loads reduced by R are multiplied by C_d before checking drift. The C_d term is used throughout the standard for fixed-base structures to approximate the ratio of actual earthquake response to response calculated for “reduced” forces. Generally, C_d

is 1/2 to 4/5 the value of R. For isolated structures, the R_I factor is used both to reduce lateral loads and to increase displacements (calculated for reduced lateral loads) before checking drift. Equivalency would be obtained if the drift limits for both fixed-base and isolated structures were based on their respective R factors. It may be noted that the drift limits for isolated structures generally are more conservative than those for conventional, fixed-base structures, even where fixed-base structures are assigned to Occupancy Category IV.

C17.6 DYNAMIC ANALYSIS PROCEDURES

This section specifies the requirements and limits for dynamic procedures. The design displacement and force limits on response spectrum and response history procedures are shown in Table C17.4-1.

A more detailed or refined study can be performed in accordance with the analysis procedures described in this section. The intent of this section is to provide procedures that are compatible with the minimum requirements of Section 17.5. Reasons for performing a more refined study include:

1. The importance of the building.
2. The need to analyze possible structure/isolation-system interaction where the fixed-base period of the building is greater than one-third of the isolated period.
3. The need to explicitly model the deformational characteristics of the lateral-force-resisting system where the structure above the isolation system is irregular.
4. The desirability of using site-specific ground-motion data, especially for very soft or liquefiable soils (Site Class F) or for structures located where S_1 is greater than 0.60.
5. The desirability of explicitly modeling the deformational characteristics of the isolation system. This is especially important for systems that have damping characteristics that are amplitude-dependent, rather than velocity-dependent, since it is difficult to determine an appropriate value of equivalent viscous damping for these systems.

Section C17.4 discusses other conditions that require use of the response history procedure. As shown in Table C17.4-1, the drift limit for isolated structures is relaxed where story drifts are computed using nonlinear response history analysis.

Where response history analysis is used as the basis for design, the design displacement of the isolation system and design forces in elements of the structure above are computed from not fewer than three separate analyses, each using a different ground motion selected and scaled in accordance with Section 17.3.2. Where the configuration of the isolation system or of the superstructure is not symmetric, additional analyses are required to satisfy the requirement of Section 17.6.3.4 to consider the most disadvantageous location of eccentric mass. As appropriate, near-field phenomena may also be incorporated. As in the nuclear industry, where at least seven ground motions are used for nonlinear response history analysis, it is considered appropriate to base design of seismically isolated structures on the average value of the response parameters of interest.

C17.7 DESIGN REVIEW

Review of the design and analysis of the isolation system and design review of the isolator testing program is mandated by the standard for two key reasons:

1. The consequences of isolator failure could be catastrophic.
2. Isolator design and fabrication technology is evolving rapidly and may be based on technologies unfamiliar to many design professionals.

The standard requires review to be performed by a team of registered design professionals that are independent of the design team and other project contractors. The review team should include individuals with special expertise in one or more aspects of the design, analysis, and implementation of seismic isolation systems.

The review team should be formed prior to the development of design criteria (including site-specific ground shaking criteria) and isolation system design options. Further, the review team should have full access to all pertinent information and the cooperation of the design team and regulatory agencies involved with the project.

C17.8 TESTING

The design displacements and forces determined using the standard assume that the deformational characteristics of the isolation system have been defined previously by comprehensive testing. If comprehensive test data are not available for a system, major design alterations in the structure may be necessary after the tests are complete. This would result from variations in the isolation-system properties assumed for design and those obtained by test. Therefore, it is advisable that prototype systems be tested during the early phases of design, if sufficient test data are not available on an isolation system.

Typical force-deflection (or hysteresis) loops are shown in Figure C17.8-1; also illustrated are the values defined in Section 17.8.5.1.

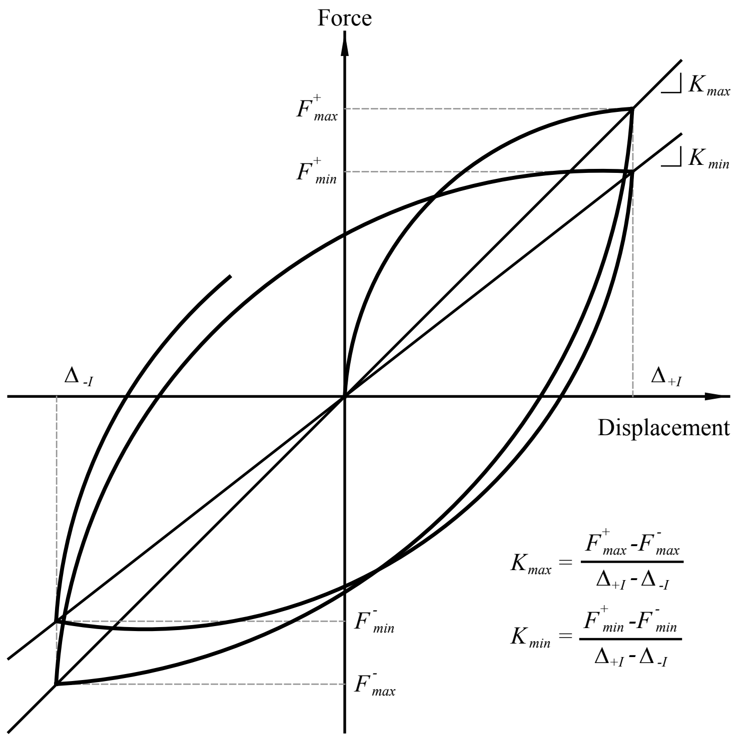

Figure C17.8-1 The effect of stiffness on an isolation bearing.

The required sequence of tests will verify experimentally the following:

1. The assumed stiffness and capacity of the wind-restraining mechanism;
2. The variation of the isolator's deformational characteristics with amplitude (and with vertical load, if it is a vertical load-carrying member);
3. The variation of the isolator's deformational characteristics for a realistic number of cycles of loading at the design displacement; and
4. The ability of the system to carry its maximum and minimum vertical loads at the maximum displacement.

Force-deflection tests are not required if similarly sized components have been tested previously using the specified sequence of tests.

The variations in the vertical loads required for tests of isolators which carry vertical, as well as lateral, load are necessary to determine possible variations in the system properties with variations in overturning force.

C17.8.5 Design Properties of the Isolation System

C17.8.5.1 Maximum and Minimum Effective Stiffness. The effective stiffness is determined from the hysteresis loops as shown in Figure C17.8-1. Stiffness may vary considerably as the test amplitude increases but should be reasonably stable (within 15 percent) for more than three cycles at a given amplitude.

The intent of these requirements is that the deformational properties used in design result in the maximum design forces and displacements. For determining design displacement, this means using the smallest damping and effective stiffness values. For determining design forces, this means using the smallest damping value and the largest stiffness value.

C17.8.5.2 Effective Damping. The determination of equivalent viscous damping is reasonably reliable for systems whose damping characteristics are velocity dependent. For systems that have amplitude-dependent energy-dissipating mechanisms, significant problems arise in determining an equivalent viscous-damping value. Since it is difficult to relate velocity- and amplitude-dependent phenomena, it is recommended that where the equivalent-viscous damping assumed for the design of amplitude-dependent energy-dissipating mechanisms (such as pure-sliding systems) is greater than 30 percent, the design-basis force and displacement be determined using the response history procedure, as discussed in Section C17.4.

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COMMENTARY TO CHAPTER 18, SEISMIC DESIGN REQUIREMENTS FOR STRUCTURES WITH DAMPING SYSTEMS

C18.1 GENERAL

The requirements of this chapter apply to all types of damping systems including both displacement-dependent damping devices of hysteretic or friction systems and velocity-dependent damping devices of viscous or visco-elastic systems (Soong and Dargush, 1997; Constantinou et al., 1998; Hanson and Soong, 2001). Compliance with these requirements is intended to produce performance comparable to that for a structure with a conventional seismic-force-resisting system, but the same methods can be used to achieve higher performance.

The damping system (DS) is defined separately from the seismic-force-resisting system (SFRS), although the two systems may have common elements. As illustrated in Figure C18.1-1, the damping system may be external or internal to the structure and may have no shared elements, some shared elements, or all elements in common with the seismic-force-resisting system. Elements common to the damping system and the seismic-force-resisting system must be designed for a combination of the two loads of the two systems. When the DS and SFRS have no common elements, the damper forces must be collected and transferred to members of the SFRS.

C18.2 GENERAL DESIGN REQUIREMENTS

C18.2.2 System Requirements. Structures with a damping system must have a seismic-force-resisting system that provides a complete load path. The seismic-force-resisting system must comply with all of the height, Seismic Design Category, and redundancy limitations and with the detailing requirements specified in this standard for the specific seismic-force-resisting system. The seismic-force-resisting system without the damping system (as if damping devices were disconnected) must be designed to have not less than 75 percent of the strength required for undamped structures having that type of seismic-force-resisting system (and not less than 100 percent if the structure is horizontally or vertically irregular). The damping systems, however, may be used to meet the drift limits (whether the structure is regular or irregular). Having the SFRS designed for a minimum of 75 percent of the strength required for undamped structures provides safety in the event of damping system malfunction and produces a composite system with sufficient stiffness and strength to have controlled lateral displacement response.

The damping system must be designed for the actual (unreduced) earthquake forces (such as, peak force occurring in damping devices) and deflections. For certain elements of the damping system (such as the connections or the members into which the damping devices frame), other than damping devices, limited yielding is permitted provided such behavior does not affect damping system function or exceed the amount permitted for elements of conventional structures by the standard.

C18.2.4 Procedure Selection. Linear static and response spectrum analysis methods can be used for design of structures with damping systems that meet certain configuration and other limiting criteria (for example, at least two damping devices at each story configured to resist torsion). In such cases, additional nonlinear response history analysis shall be used to confirm peak responses when the structure is located at a site with S_1 greater than or equal to 0.6. The analysis methods damperd structures are based on nonlinear static “pushover” characterization of the structure and calculation of peak response using effective (secant) stiffness and effective damping properties of the first (pushover) mode in the direction of interest. These are the concepts used in Chapter 17 to characterize the force-deflection properties of isolation systems, modified to incorporate explicitly the effects of ductility demand (post-yield response) and higher-mode response of structures with dampers. Like conventional structures, dampered structures generally yield during strong ground shaking, and their performance can be influenced strongly by response of higher modes.

The response spectrum and equivalent lateral force procedures presented in the standard have several simplifications and limits, as outlined below:

1. A multi-degree-of-freedom (MDOF) structure with a damping system can be transformed into equivalent single-degree-of-freedom (SDOF) systems using modal decomposition procedures. This assumes that the collapse mechanism for the structure is a single-degree-of-freedom mechanism so that the drift distribution over height can be estimated reasonably using either the first mode shape or another profile, such as an inverted triangle. Such procedures do not strictly apply to either yielding buildings or buildings that are non-proportionally damped.
2. The response of an inelastic single-degree-of-freedom system can be estimated using equivalent linear properties and a 5-percent-damped response spectrum. Spectra for damping greater than 5 percent can be established using damping

coefficients, and velocity-dependent forces can be established either by using the pseudo-velocity and modal information or by applying correction factors to the pseudo-velocity.

3. The nonlinear response of the structure can be represented by a bilinear hysteretic relationship with zero post-elastic stiffness (elasto-plastic behavior).
4. The yield strength of the structure can be estimated either by performing simple plastic analysis or by using the specified minimum seismic base shear and values of R, Ω_0, and C_d.
5. Higher modes need to be considered in the equivalent lateral force procedure in order to capture their effects on velocity-dependent forces. This is reflected in the residual mode procedure.

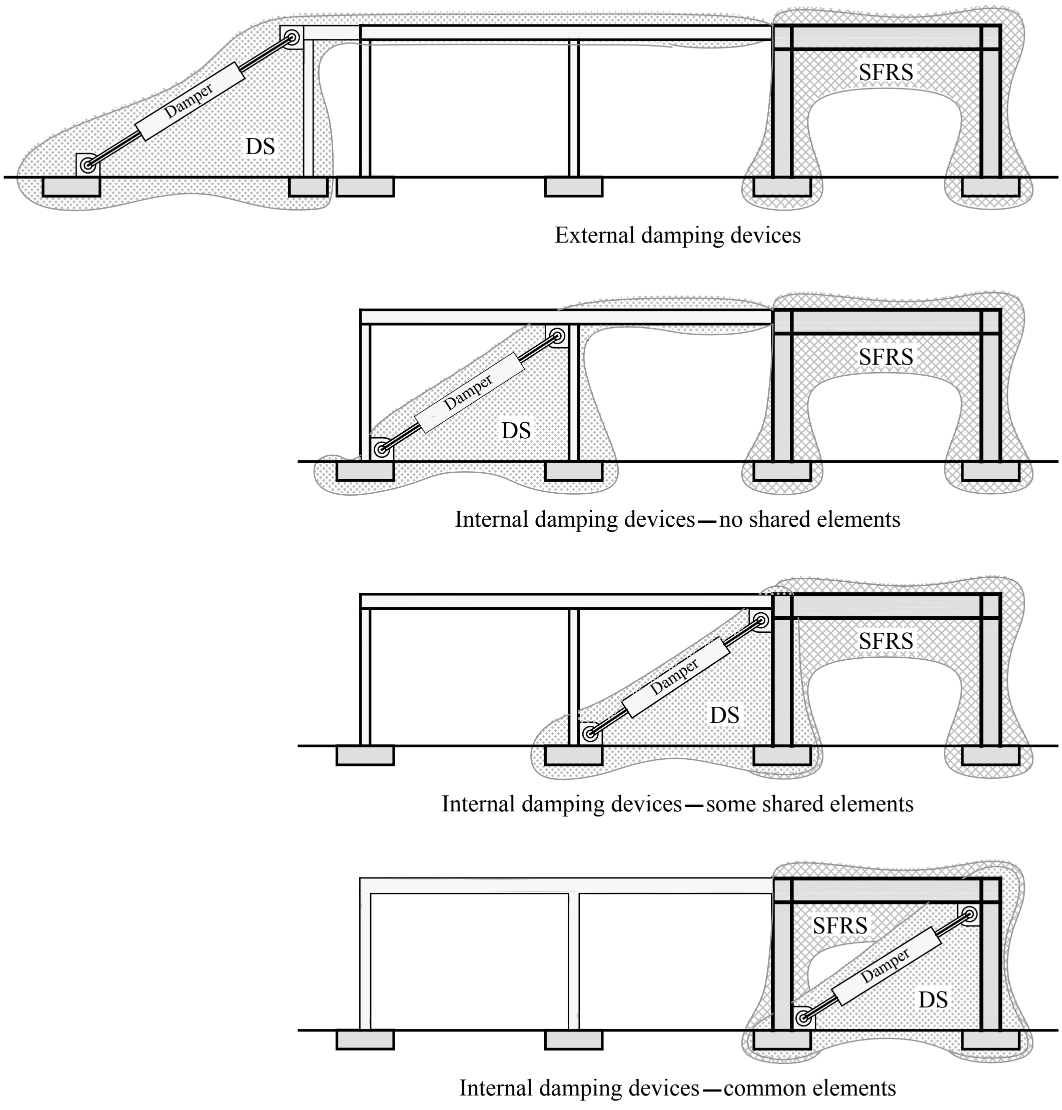

Figure C18.1-1 Damping system (DS) and seismic-force-resisting system (SFRS) configurations.

FEMA 440 (Applied Technology Council, 2005) presents a review of simplified procedures for the analysis of yielding structures. The combined effects of the simplifications mentioned above are reported by Ramirez et al. (2001) and Pavlou and Constantinou (2004) based on studies of 3-story and 6-story buildings with damping systems designed by the procedures of the standard. The response spectrum and equivalent lateral force procedures of the standard are found to provide conservative predictions of drift and predictions of damper forces and member actions that are of acceptable accuracy when compared to results of nonlinear dynamic response history analysis. When designed in accordance with the standard, structures with damping systems are expected to have structural performance at least as good as that of structures without damping systems. Pavlou and Constantinou (2006) report that structures with damping systems designed in accordance with the standard provide the benefit of reduced secondary system response, although this benefit is restricted to systems with added viscous damping.

C18.3 NONLINEAR PROCEDURES

For designs in which the seismic-force-resisting-system is essentially elastic (assuming an overstrength of 50 percent), the only nonlinear characteristics that must be modeled in the analysis are those of the damping devices. For designs in which the seismic-force-resisting system will yield, the post-yield behavior of the structural elements must be modeled explicitly.

C18.4 RESPONSE SPECTRUM PROCEDURES and C18.5 EQUIVALENT LATERAL FORCE PROCEDURE

Effective Damping

In the standard the reduced response of a structure with a damping system is characterized by the damping coefficient, B, based on the effective damping, β, of the mode of interest. This is the same approach as that used for isolated structures. Like isolation, effective damping of the fundamental-mode of a damped structure is based on the nonlinear force-deflection properties of the structure. For use with linear analysis methods, nonlinear properties of the structure are inferred from the overstrength factor, Ω_0, and other terms.

Figure C18.4-1 illustrates reduction in design earthquake response of the fundamental mode due to increased effective damping (represented by coefficient, B_{ID}). The capacity curve is a plot of the nonlinear behavior of the fundamental mode in spectral acceleration-displacement coordinates. The reduction due to damping is applied at the effective period of the fundamental mode of vibration (based on the secant stiffness).

In general, effective damping is a combination of three components:

1. Inherent Damping (β_I)—Inherent damping of the structure at or just below yield, excluding added viscous damping (typically assumed to be 5 percent of critical for structural systems without dampers).
2. Hysteretic Damping (β_H)—Post-yield hysteretic damping of the seismic-force-resisting system and elements of the damping system at the amplitude of interest (taken as 0 percent of critical at or below yield).
3. Added Viscous Damping (β_V)—Viscous component of the damping system (taken as 0 percent for hysteretic or friction-based damping systems).

Both hysteretic damping and added viscous damping are amplitude-dependent, and the relative contributions to total effective damping change with the amount of post-yield response of the structure. For example, adding dampers to a structure decreases post-yield displacement of the structure and, hence, decreases the amount of hysteretic damping provided by the seismic-force-resisting system. If the displacements are reduced to the point of yield, the hysteretic component of effective damping is zero, and the effective damping is equal to inherent damping plus added viscous damping. If there is no damping system (as in a conventional structure), effective damping simply equals inherent damping (typically assumed to be 5 percent of critical for most conventional structures).

Linear Analysis Methods

The section specifies design earthquake displacements, velocities, and forces in terms of design earthquake spectral acceleration and modal properties. For equivalent lateral force (ELF) analysis, response is defined by two modes: the fundamental mode and the residual mode. The residual mode is a new concept used to approximate the combined effects of higher modes. While typically of secondary importance to story drift, higher modes can be a significant contributor to story velocity and, hence, are important for design of velocity-dependent damping devices. For response spectrum analysis, higher modes are explicitly evaluated.

For both the ELF and the response spectrum analysis procedures, response in the fundamental mode in the direction of interest is based on assumed nonlinear (pushover) properties of the structure. Nonlinear (pushover) properties, expressed in terms of base shear and roof displacement, are related to building capacity, expressed in terms of spectral coordinates, using mass participation and other fundamental-mode factors shown in Figure C18.4-2. The conversion concepts and factors shown in Figure C18.4-2 are the same as those defined in Chapter 9 of *Seismic Rehabilitation of Existing Buildings* (ASCE/SEI 41), which addresses seismic rehabilitation of a structure with damping devices.

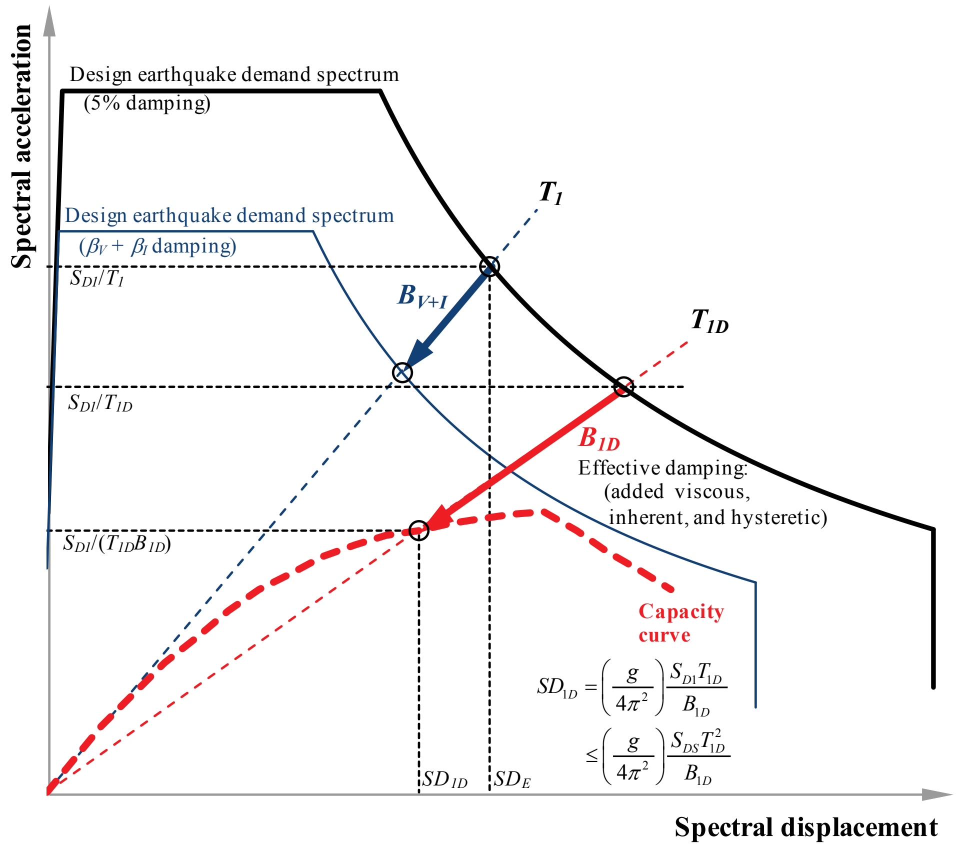

Figure C18.4-1 Effective damping reduction of design demand.

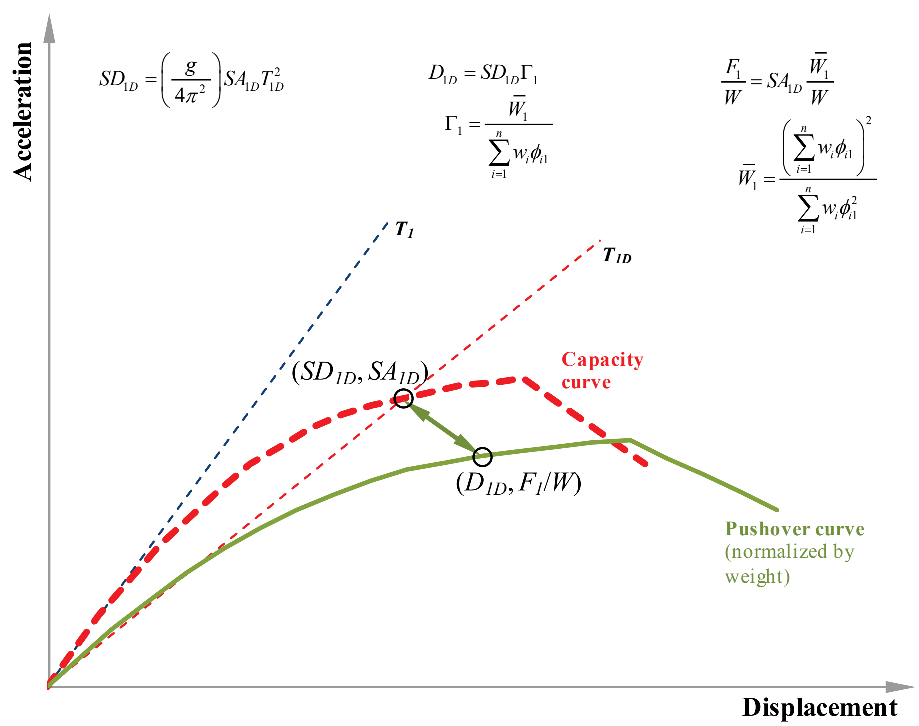

Figure C18.4-2 Pushover and capacity curves.

Where using linear analysis methods, the shape of the fundamental-mode pushover curve is not known, so an idealized elasto-plastic shape is assumed, as shown in Figure C18.4-3. The idealized pushover curve is intended to share a common point with the actual pushover curve at the design earthquake displacement, D_{1D}. The idealized curve permits definition of the global ductility demand due to the design earthquake, μ_D, as the ratio of design displacement, D_{1D}, to yield displacement, D_Y. This ductility factor is used to calculate various design factors; it must not exceed the ductility capacity of the seismic-force-resisting system, μ_{max}, which is calculated using factors for conventional structural response. Design examples using linear analysis methods have been developed and found to compare well with the results of nonlinear time history analysis (Ramirez et al., 2001).

Elements of the damping system are designed for fundamental-mode design earthquake forces corresponding to a base shear value of V_Y (except that damping devices are designed and prototypes tested for maximum considered earthquake response). Elements of the seismic-force-resisting system are designed for reduced fundamental-mode base shear, V_1, where force reduction is based on system overstrength (represented by Ω_0), multiplied by C_d/R for elastic analysis (where actual pushover strength is not known). Reduction using the ratio C_d/R is necessary because the standard provides values of C_d that are less than those for R. Where the two parameters have equal value and the structure is 5 percent damped under elastic conditions, no adjustment is necessary. Because the analysis methodology is based on calculating the actual story drifts and damping device displacements (rather than the displacements calculated for elastic conditions at the reduced base shear and then multiplied by C_d), an adjustment is needed. Since actual story drifts are calculated, the allowable story drift limits of Table 12.12-1 are multiplied by R/C_d before use.

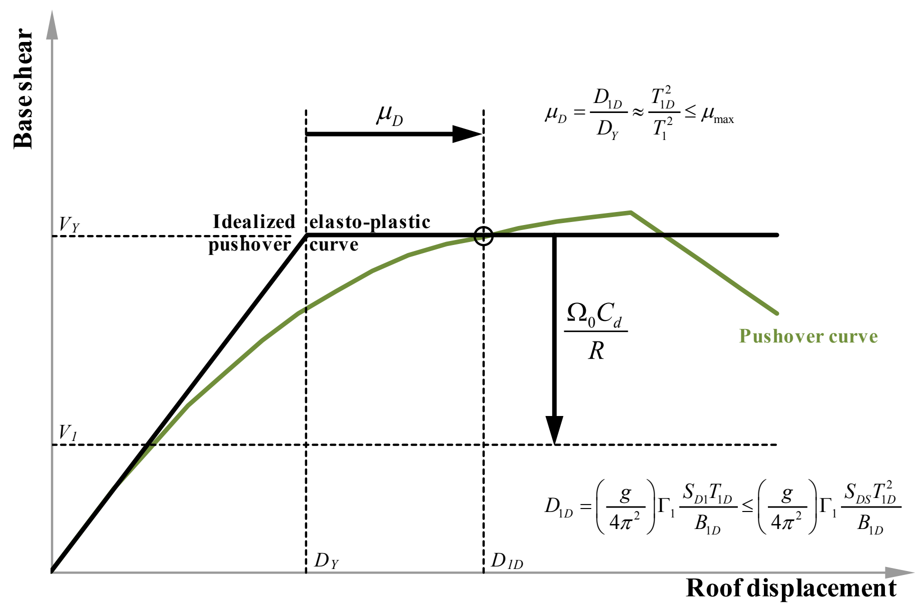

Figure C18.4-3 Idealized elasto-plastic pushover curve used for linear analysis.

C18.6 DAMPED RESPONSE MODIFICATION

C18.6.1 Damping Coefficient

Values of the damping coefficient, B, in Table 18.6-1 for design of damped structures are the same as those in Table 17.5-1 for isolated structures at damping levels up to 20 percent, but extend to higher damping levels based on results presented in Ramirez et al. (2001). Table C18.6-1 compares values of the damping coefficient as found in the standard and various resource documents and codes. FEMA 440 and the draft of Eurocode 8 present equations for the damping coefficient, B, whereas the other documents present values of B in tabular format.

The equation in FEMA 440 is

$$B = \frac{4}{5.6 - \ln(100\beta)}$$

The equation in Eurocode 8 is

$$B = \sqrt{\frac{0.05 + \beta}{0.10}}$$

Table C18.6-1 Values of Damping Coefficient, *B*

Effective Damping, β (%)	Table 17.5-1, 1999 AASHTO, 2001 CBC (seismically isolated structures)	Table 18.6-1 (structures with damping systems)	FEMA 440	EUROCODE 8
2	0.8	0.8	0.8	0.8
5	1.0	1.0	1.0	1.0
10	1.2	1.2	1.2	1.2
20	1.5	1.5	1.5	1.6
30	1.7	1.8	1.8	1.9
40	1.9	2.1	2.1	2.1
50	2.0	2.4	2.4	2.3

C18.6.2 Effective Damping. The effective damping is calculated assuming the structural system exhibits perfectly bi-linear hysteretic behavior characterized by the effective ductility demand, μ, as described in Ramirez et al. (2001). Effective damping is adjusted using the hysteresis loop adjustment factor, q_H, which is the actual area of the hysteresis loop divided by the area of the assumed perfectly bi-linear hysteretic loop. In general, values of this factor are less than unity. In Ramirez et al. (2001) expressions for this factor (which they call Quality Factor) are too complex to serve as a simple rule. Equation 18.6-5 provides a simple estimate of this factor. The equation predicts correctly the trend in the constant acceleration domain of the response spectrum, and it is believed to be conservative for flexible structures.

C18.7 SEISMIC LOAD CONDITIONS AND ACCEPTANCE CRITERIA

C18.7.2.5 Seismic Load Conditions and Combination of Modal Responses. Seismic design forces in elements of the damping system are calculated at three distinct stages: maximum displacement, maximum velocity, and maximum acceleration. All three stages need to be checked for structures with velocity-dependent damping systems. For displacement-dependent damping systems, the first and third stages are identical, whereas the second stage is inconsequential.

Force coefficients C_{mFD} and C_{mFV} are used to combine the effects of forces calculated at the stages of maximum displacement and maximum velocity to obtain the forces at maximum acceleration. The coefficients are presented in tabular form based on analytic expressions presented in Ramirez et al. (2001) and account for nonlinear viscous behavior and inelastic structural system behavior.

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COMMENTARY FOR CHAPTER 19, SOIL STRUCTURE INTERACTION FOR SEISMIC DESIGN

C19.1 GENERAL

The response of a structure to earthquake shaking is affected by interactions between three linked systems: the structure, the foundation, and the geologic media underlying and surrounding the foundation. A seismic Soil-Structure Interaction (SSI) analysis evaluates the collective response of these systems to a specified free-field ground motion. The term "free-field" refers to motions not affected by structural vibrations and represents the condition for which the design spectrum is derived using the procedures given in Chapter 11.

SSI effects are absent for the theoretical condition of rigid foundation and soil conditions. Accordingly, SSI effects reflect the differences between the actual response of the structure and the response for the theoretical, rigid base condition. Visualized within this context, three SSI effects can significantly affect the response of building structures:

1. Foundation stiffness and damping. Inertia developed in a vibrating structure gives rise to base shear, moment, and torsional excitation, and these loads in turn cause displacements and rotations of the foundation relative to the free field. These relative displacements and rotations are only possible because of compliance in the soil-foundation system, which can significantly contribute to the overall structural flexibility in some cases. Moreover, the relative foundation-free field motions give rise to energy dissipation via radiation damping (i.e., damping associated with wave propagation into the ground away from the foundation, which acts as the wave source) and hysteretic soil damping, and this energy dissipation can significantly affect the overall damping of the soil-foundation-structure system. Since these effects are rooted in the structural inertia, they are referred to as inertial interaction effects.
2. Variations between free-field and foundation-level ground motions. The differences between foundation and free-field motions result from two processes. The first is known as kinematic interaction and results from the presence of stiff foundation elements on or in soil, which cause foundation motions to deviate from free-field motion as a result of base slab averaging, wave scattering, and embedment effects. Procedures for modifying design spectra to account for these effects are given in FEMA 440 and ASCE/SEI 41. The second process is related to the structure and foundation inertia and consists of the relative foundation-free field displacements and rotations described above.
3. Foundation deformations. Flexural, axial, and shear deformations of foundation elements occur as a result of loads applied by the superstructure and the supporting soil medium. Such deformations represent the seismic demand for which foundation components should be designed. These deformations can also significantly affect the overall system behavior, especially with respect to damping.

Chapter 19 treats only the inertial interaction effects (the first item above). Inertial interaction in buildings tends to be important for stiff structural systems (such as shear walls and braced frames), particularly where the foundation soil is relatively soft (i.e., Site Classes C to E). Kinematic interaction effects are neglected in these provisions. Foundation design is covered in Section 12.13.

The procedures in Chapter 19 are used to modify the fixed-base properties (period and damping) of a structural system. If fixed-base properties are obtained from an analytical model of the structure, the fixed-base properties correspond to a condition without soil springs. If soil springs are included in the analytical model of the structure, then the procedures given in Chapter 19 should not be used to modify the building period. The damping procedures in Chapter 19 could still be used in this case if the foundation springs are linear (thus introducing no damping) and there are no dashpots in parallel with the springs. In the remainder of this commentary, it is assumed that the structural period and damping ratio that are being modified for SSI effects correspond to a fixed-base condition.

In design procedures that utilize response spectra to establish design values of base shear (i.e., force-based methods such as those given in Chapter 12), the effects of inertial SSI on the seismic response of buildings is represented as a function of the ratio of flexible- to fixed-base first-mode natural period, $\tilde{T}_1/T_1$, and system damping, β_0, attributable to foundation-soil interaction. The flexible-base first-mode damping ratio, $\tilde{\beta}$, is calculated using Equation 19-9. Figure C19-1 illustrates two possible effects of inertial SSI on the peak base shear, which is commonly computed from spectral acceleration at the first-mode. The spectral acceleration for a flexible-based structure ($\tilde{S}_a = \tilde{C}_s/g$) is obtained by entering the spectrum drawn for effective damping ratio, $\tilde{\beta}$, at the corresponding elongated period, $\tilde{T}$. For buildings with periods greater than about 0.5 s, using $\tilde{S}_a$ in lieu of S_a (=C_s/g) typically reduces base shear demand, whereas in very stiff structures SSI can increase the base shear. Most equivalent lateral force methods feature a flat spectral shape at low periods that, when coupled with the requirement that $\tilde{\beta} > \beta$, results in modeling of inertial SSI that can only decrease the base shear demand.

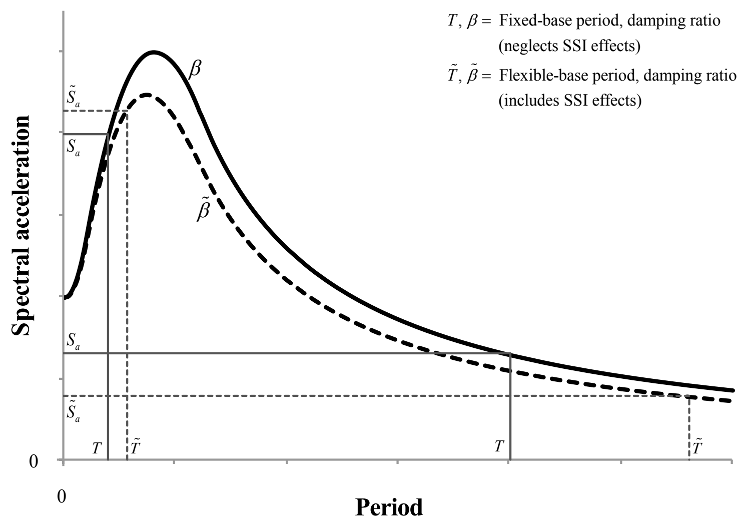

Figure C19-1 Schematic showing effects of period lengthening and foundation damping on design spectral accelerations.

The method given in Chapter 19 for evaluating inertial SSI effects is optional and has rarely been used in practice. There are several reasons for this. First, because the guidelines were written such that base shear demand can only decrease through consideration of SSI, SSI effects are ignored in order to be conservative. Second, many design engineers who have attempted to apply the method on projects have done so for major, high-rise buildings for which they felt evaluating SSI effects could provide cost savings. Unfortunately, inertial interaction effects are negligible for these tall, flexible structures, and hence the design engineers realized no benefit for their efforts and thereafter discontinued use of the procedures. The use of the procedures actually yield the most benefit for short-period ($T < 1$ sec), stiff structures with stiff, interconnected foundation systems (i.e., mats or interconnected footings) founded on soil.

C19.2 EQUIVALENT LATERAL FORCE PROCEDURE

This procedure considers the response of the structure in its fundamental mode of vibration and accounts for the contributions of the higher modes to story shears implicitly through the choice of the effective weight of the structure and the vertical distribution of the lateral forces. The effects of soil-structure interaction are accounted for on the assumption that they influence only the contribution of the fundamental mode of vibration.

C19.2.1 Base Shear. Base shear is reduced for the effects of SSI as indicated in Equation 19.2-1 and 19.2-2. As indicated in Equation 19.2-2, the change in base shear is related to the change in seismic coefficient (or spectral acceleration, as shown in Figure C19-1). The term $(0.05\,\tilde{\beta})^{0.4}$ in Equation 19.2-2 represents the reduction in spectral ordinate associated with a change of damping from the fixed base value of $\beta = 0.05$ to the flexible base value of $\tilde{\beta}$.

C19.2.1.1 Effective Building Period. The fixed base period, T, is lengthened to the flexible-base period, $\tilde{T}$, using Equation 19.2-3, which was derived by Veletsos and Meek (1974). Terms K_y and K_θ represent the translational and rocking stiffnesses of the foundation, respectively. The standard does not provide guidance on the evaluation of these stiffness terms. Equations for K_y and K_θ are given by Gazetas (1991), and a number of practical considerations associated with the analysis of these terms are reviewed in FEMA 440 (2005). For convenience, simplified guidelines are presented below for these stiffness terms for use with the standard.

For building foundation systems having lateral continuity, such as mats or footings interconnected with grade beams, stiffnesses K_y and K_θ can often be approximated as:

$$K_y = \frac{8}{2-\nu} G r_a \qquad \text{(C19-2)}$$

$$K_\theta = \frac{8}{3(1-\nu)} G r_m^3 \alpha_\theta \qquad \text{(C19-3)}$$

where: r_a = an equivalent foundation radius that matches the area of the foundation, A_0 (i.e., $r_a = \sqrt{(A_0/\pi)}$); r_m = an equivalent foundation radius that matches the moment of inertia of the foundation in the direction of shaking (i.e., $r_m = \sqrt[4]{4I_0/\pi}$); G = the strain-dependent shear modulus, as defined in the standard; ν = the soil Poisson's ratio (generally taken as 0.3 for sands and 0.45 for clays); and α_θ = a dimensionless coefficient that depends on the period of excitation, the dimensions of the foundation, and the properties of the supporting medium (Luco, 1974; Veletsos and Verbic, 1973; Veletsos and Wei, 1971). A similar coefficient exists for translation (α_y), but can be taken as 1.0 without introducing significant error, and hence is not shown in Equation C19-2.

As noted in the standard, shear modulus G is evaluated from small-strain shear wave velocity as $G = (G/G_o)G_o = (G/G_o)\gamma v^2_{so}/g$ (all terms defined in the standard). Shear wave velocity, v_{s0}, should be evaluated as the average small-strain shear wave velocity within the effective depth of influence below the foundation. The effective depth should be taken as $0.75r_a$ for horizontal vibrations of the foundation and $0.75r_m$ for rocking vibrations (Stewart et al., 2003). Methods for measuring v_{s0} (preferred) or estimating it from other soil properties are summarized elsewhere (e.g., Kramer, 1996).

The dynamic modifier for rocking, α_θ, can significantly affect the computed response of some building structures. In the absence of more detailed analyses, for ordinary building structures with an embedment ratio $d/r_m < 0.5$ (where d = depth of embedment, measured from ground surface to base of foundation), the factor α_θ can be estimated as follows (Stewart et al., 2003):

$r_m/(v_{s0}T)$	α_θ
< 0.05	1.0
0.15	0.85
0.35	0.7
0.5	0.6

Foundation embedment has the effect of increasing the stiffnesses K_y and K_θ. For embedded foundations for which there is positive contact between the side walls and the surrounding soil, K_y and K_θ may be determined from the following approximate formulas (Kausel, 1974):

$$K_y = \frac{8Gr_a}{2-\nu}\left[1+\left(\frac{2}{3}\right)\left(\frac{d}{r_a}\right)\right] \qquad \text{(C19-4)}$$

$$K_\theta = \frac{8Gr_m^3}{3(1-\nu)}\left[1+2\left(\frac{d}{r_m}\right)\right] \qquad \text{(C19-5)}$$

Experimental studies and field performance data (Stokoe and Erden, 1975; Stewart et al., 1999) indicate that the effects of foundation embedment are sensitive to the condition of the backfill and that judgment must be exercised in using Equations C19-4 and C19-5. For example, if contact is lost between the soil and basement walls, stiffnesses K_y and K_θ should be determined from the formulas for surface-supported foundations. More generally, the quantity d above should be interpreted as the effective depth of foundation embedment for the conditions that would prevail during the design earthquake ground motion.

The formulas for K_y and K_θ presented above can be applied to most soil profiles in which soil shear wave velocity, v_{s0}, changes with depth. However, if the soil profile consists of a surface stratum of soil underlain by a much stiffer deposit with a shear wave velocity more than twice that of the surface layer, K_y and K_θ may be determined from the following two generalized formulas in which G is the shear modulus of the surface soil and D_s is the total depth of the stratum:

$$K_y = \frac{8Gr_a}{2-\nu}\left[1+\left(\frac{2}{3}\right)\left(\frac{d}{r_a}\right)\right]\left[1+\left(\frac{1}{2}\right)\left(\frac{r_a}{D_s}\right)\right]\left[1+\left(\frac{5}{4}\right)\left(\frac{d}{D_s}\right)\right] \qquad \text{(C19-6)}$$

$$K_\theta = \frac{8Gr_m^3}{3(1-\nu)}\left[1+2\left(\frac{d}{r_m}\right)\right]\left[1+\left(\frac{1}{6}\right)\left(\frac{r_m}{D_s}\right)\right]\left[1+0.7\left(\frac{d}{D_s}\right)\right] \qquad \text{(C19-7)}$$

The above formulas are based on analyses of a stratum supported on a rigid base (Elsabee et al., 1977; Kausel and Roesset, 1975) and apply for $r/D_s < 0.5$ and $d/r < 1$ (r taken as either r_a or r_m). The applicability of those rigid base solutions to practical situations (non-rigid geologic media) was evaluated by Stewart et al. (2003), resulting in the recommendations provided above.

For buildings supported on footing foundations, the above formulas can generally be used with r_a and r_m calculated using the full building footprint dimensions, provided that the footings are interconnected with grade beams. An exception can occur for buildings with both shear walls and frames, for which the rotation of the foundation beneath the wall may be independent of that for the foundation beneath the column (this is referred to as weak rotational coupling). In such cases, r_m is often best calculated using the dimensions of the wall footing. Very stiff foundations, which provide strong rotational coupling, are best described using an r_m value that reflects the full foundation dimension. Regardless of the degree of rotational coupling, r_a should be calculated using the full foundation dimension if foundation elements are interconnected or continuous. Further discussion can be found in FEMA (2005). The use of discrete (non-interconnected) spread footing foundations in seismic regions is not recommended.

For buildings supported on pile foundations, lateral stiffness, K_y, can be taken as the sum of the lateral head stiffnesses of the supporting piles. These stiffness values are generally calculated using a beam on Winkler foundation model, which is discussed in detail elsewhere (e.g., Salgado, 2006). Rotational stiffness, K_θ, can be calculated from the vertical stiffness of the individual piles, k_{zi}, as follows:

$$K_\theta \approx \sum_i k_{zi} y_i^2 \qquad \text{(C19-8)}$$

where y_i = horizontal distance from the foundation centroidal axis to pile i measured in the direction of shaking. The approximation in Equation C19-8 assumes an infinitely rigid pile cap and neglects the rotational stiffness of individual piles, which is typically a small contribution. Quantity k_{zi} can be calculated for an individual pile using well-established methods, such as discrete element modeling with *t-z* curves (e.g., Salgado, 2006).

The alternate approach in the standard, represented by Equation 19.2-5, was derived using Poisson's ratio $\nu = 0.4$, and is generally sufficient for non-embedded foundations that are laterally continuous across the building footprint and for which there is no "rigid" layer at depth in the profile (which would require the use of Equations C19-6 and C19-7 to calculate foundation stiffness). The value of relative weight parameter, α (defined in the standard), can be taken as approximately 0.15 for typical buildings.

C19.2.1.2 Effective Damping. Bielak (1975, 1976) and Veletsos and Nair (1975) expressed the flexible-base first-mode damping ratio, $\tilde{\beta}$, as indicated in Equation 19.2-9. This equation is based on analyses of the harmonic response of single-degree-of-freedom oscillators supported on a visco-elastic medium with hysteretic damping. Foundation damping factor β_0 incorporates the effects of energy dissipation into the soil due to radiation damping and hysteretic damping in the soil.

Figure 19.2-1 shows β_0 as a function of period lengthening ratio and was derived from the analytical solution presented in Veletesos and Nair (1975) for the condition of zero foundation embedment. Additional damping can be realized for embedded foundations, and the use of damping values from Figure 19.2-1 is conservative for such conditions. More exact solutions can be obtained using procedures given in FEMA (2005).

Equation 19.2-9, in combination with the information presented in Figure 19.2-1, may lead to damping factors for the soil-foundation-structure system, $\tilde{\beta}$, that are smaller than the fixed base structural damping, β (assumed to be 0.05). However, it is recommended that $\tilde{\beta}$ never be taken as less than 0.05 for design applications. The use of values of $\tilde{\beta} > \beta$ is well-justified from field case-history data (Stewart et al., 1999, 2003).

The presence of a stiff layer at depth in the soil profile can impede radiation damping, rendering the values in Figure 19.2-1 too high. If a site consists of a relatively uniform layer of depth, D_s, overlying a very stiff layer with a shear wave velocity more than twice that of the surface layer, damping values should be reduced as indicated by Equation 19.2-12.

C19.2.2 Vertical Distribution of Seismic Forces. The vertical distributions of the equivalent lateral forces for flexibly and rigidly supported structures are similar, and it is recommended that the same distribution be used in both cases, changing only the magnitude of the forces to correspond to the appropriate base shear. A greater degree of refinement in this step would be inconsistent with the approximations embodied in the requirements for rigidly supported structures.

With the vertical distribution of the lateral forces established, the overturning moments and the torsional effects about a vertical axis are computed as for rigidly supported structures. The above procedure is applicable to planar structures and, with some extension, to three-dimensional structures.

C19.2.3 Other Effects. In addition to its effect on base shear, inertial SSI also can increase the horizontal displacements of the structure relative to its base (because of rocking). This can increase the required spacing between structures and secondary design forces associated with *P*-delta effects. Such effects can be significant for stiff structural systems (e.g., walls and braced frames).

C19.3 MODAL ANALYSIS PROCEDURE

The procedure outlined above in Section C19.2 is applicable to a modal analysis by adjusting the modal period and damping ratio of the fundamental mode only. Higher modes are relatively unaffected by SSI (e.g., Bielak, 1976; Chopra and Gutierrez, 1974; Veletsos, 1977). Hence, the contributions of higher modes are computed as if the structure were fixed at the base, and the maximum value of a response quantity is determined as for fixed-base structures but with the adjusted first-mode responses.

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COMMENTARY FOR CHAPTER 20, SITE CLASSIFICATION PROCEDURE FOR SEISMIC DESIGN

C20.1 SITE CLASSIFICATION

Site classification procedures are given in Chapter 20 for the purpose of classifying the site and determining site coefficients and site-adjusted maximum considered earthquake ground motions in accordance with Section 11.4.3. Site classification procedures are also used to define the site conditions for which site-specific site response analyses are required to obtain site ground motions in accordance with Section 11.4.7 and Chapter 21.

C20.3 SITE CLASS DEFINITIONS

C20.3.1 Site Class F. Site conditions for which the site coefficients F_a and F_v in Tables 11.4-1 and 11.4-2 may not be applicable and for which site-response analyses are required by Section 11.4.7. For short-period structures it is permissible to determine values of F_a and F_v assuming that liquefaction does not occur, because ground motion data obtained in liquefied soil areas during earthquakes indicate that short-period ground motions generally are attenuated due to liquefaction whereas long-period ground motions may be amplified. This exception does not affect the requirements in Section 11.8 to assess liquefaction potential as a geologic hazard and to develop hazard mitigation measures as required.

C20.3.2 through C20.3.5. These sections and Table 20.3-1 provide definitions for Site Classes A through E. Except for the additional definitions for Site Class E in Section 20.3.2, the site classes are defined fundamentally in terms of the average small-strain shear wave velocity in the top 100 feet (30 meters) of the soil or rock profile. If shear wave velocities are available for the site, they should be used to classify the site. However, recognizing that in many cases shear wave velocities are not available for the site, alternative definitions of the site classes also are included. These definitions are based on geotechnical parameters: standard penetration resistance for cohesionless soils and rock, and standard penetration resistance and undrained shear strength for cohesive soils. The alternative definitions are intended to be conservative since the correlation between site coefficients and these geotechnical parameters is more uncertain than the correlation with shear wave velocity. That is, values of F_a and F_v will tend to be smaller if the site class is based on shear wave velocity rather than on the geotechnical parameters. Also, the site class definitions should not be interpreted as implying any specific numerical correlation between shear-wave velocity and standard penetration resistance or undrained shear strength.

Although the site class definitions in Sections 20.3.2 through 20.3.5 are straightforward, there are aspects of these assessments that may require additional judgment and interpretation. Highly variable subsurface conditions beneath a building footprint could result in overly conservative or unconservative site classification. Isolated soft soil layers within an otherwise firm soil site may not affect the overall site response if the predominant soil conditions do not include such strata. Conversely, site response studies have shown that continuous, thin, soft clay strata may affect the site amplification.

The site class should reflect the soil conditions that will affect the ground motion input to the structure or a significant portion of the structure. For structures receiving substantial ground motion input from shallow soils (for example, structures with shallow spread footings, with laterally flexible piles, or with basements where substantial ground motion input to the structure may come through the side walls), it is reasonable to classify the site on the basis of the top 100 feet (30 meters) of soils below the ground surface. Conversely, for structures with basements supported on firm soils or rock below soft soils, it may be reasonable to classify the site on the basis of the soils or rock below the mat, if it can be justified that the soft soils contribute very little to the response of the structure.

Buildings on sites with sloping bedrock or having highly variable soil deposits across the building area require careful study since the input motion may vary across the building (for example, if a portion of the building is on rock and the rest is over weak soils). Site-specific studies including two- or three-dimensional modeling may be used in such cases to evaluate the subsurface conditions and site and superstructure response. Other conditions that may warrant site-specific evaluation include the presence of low shear wave velocity soils below a depth of 100 feet (30 meters), location of the site in a sedimentary basin, or subsurface or topographic conditions with strong two- and three-dimensional site-response effects. Individuals with appropriate expertise in seismic ground motions should participate in evaluations of the need for and nature of such site-specific studies.

C20.4 DEFINITION OF SITE CLASS PARAMETERS

Section 20.4 provides formulas for defining Site Classes in accordance with definitions in Section 20.3 and Table 20.3-1. Equation 20.4-1 is for determining the effective average small-strain shear-wave velocity, $\overline{v}_s$, to a depth of 100 feet (30 meters) at a site. This equation defines $\bar{v}_s$ as 100 feet (30 meters) divided by the sum of the times for a shear wave to travel through each layer within the upper 100 feet (30 meters), where travel time for each layer is calculated as the layer thickness divided by the small-strain shear wave velocity for the layer. It is important that this method of averaging be used as it may result in a significantly lower effective average shear wave velocity than the velocity that would be obtained by directly averaging the velocities of the individual layers.

For example, consider a soil profile having four 25-foot-thick layers with shear wave velocities of 500, 1,000, 1,500, and 2,000 ft/s. The arithmetic average of the shear wave velocities is 1250 ft/s (corresponding to Site Class C), but Equation 20.4-1 produces a value of 960 ft/s (corresponding to Site Class D). The Equation 20.4-1 value is appropriate as the four layers are being represented by one layer with the same wave passage time.

Equation 20.4-2 is for classifying the site using the average standard penetration resistance blow count, $\bar{N}$, for cohesionless soils, cohesive soils, and rock in the upper 100 feet (30 meters). A method of averaging analogous to the method of Equation 20.4-1 for shear wave velocity is used. The maximum value of N that may be used for any depth of measurement in soil or rock is 100 blows/foot. For the common situation where rock is encountered, the standard penetration resistance, N, for rock layers is taken as 100.

Equations 20.4-3 and 20.4-4 are for classifying the site using the standard penetration resistance of cohesionless soil layers, N_{ch}, and the undrained shear strength of cohesive soil layers, s_u, within the top 100 feet (30 meters). These equations are provided as an alternative to using Equation 20.4-2 for which N-values in all geologic materials in the top 100 feet (30 meters) are used. Where using Equations 20.4-3 and 20.4-4, only the respective thicknesses of cohesionless soils and cohesive soils within the top 100 feet (30 meters) are used.

COMMENTARY FOR CHAPTER 21, SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN

GENERAL

Site-specific procedures for computing earthquake ground motions include dynamic site response analyses and probabilistic and deterministic seismic hazard analyses (PSHA and DSHA), which may include dynamic site response analysis as part of the calculation. Use of site-specific procedures may be required in lieu of the general procedure in Sections 11.4.1 through 11.4.6; Section C11.4.7 explains the conditions under which the use of these procedures is required. Such studies must be comprehensive and incorporate current scientific interpretations. Because there is typically more than one scientifically credible alternative for models and parameter values used to characterize seismic sources and ground motions, it is important to formally incorporate these uncertainties in a site-specific analysis. For example, uncertainties may exist in seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; ground-motion attenuation; local site conditions, including soil layering and dynamic soil properties; and possible two- or three-dimensional wave-propagation effects. The use of peer review for a site-specific ground-motion analysis is encouraged.

Site-specific ground-motion analysis can consist of one of the following approaches: (a) PSHA and possibly DSHA if the site is near an active fault, (b) PSHA/DSHA followed by dynamic site-response analysis, and (c) dynamic site response analysis only. The first approach is used to compute ground motions for bedrock or stiff soil conditions (not softer than Site Class D). In this approach, if the site consists of stiff soil overlying bedrock, for example, the analyst has the option of either (a) computing the bedrock motion from the PSHA/DSHA and then using the site-coefficient (F_a and F_v) tables in Section 11.4.3 to adjust for the stiff soil overburden or (b) computing the response spectrum at the ground surface directly from the PSHA/DSHA. The latter requires the use of attenuation equations for computing stiff soil-site response spectra (instead of bedrock response spectra).

The second approach is used where softer soils overlie the bedrock or stiff soils. The third approach assumes that a site-specific PSHA/DSHA is not necessary, but that a dynamic site response analysis should or must be performed. This analysis requires the definition of an outcrop ground motion, which can be based on the 5 percent damped response spectrum computed from the PSHA/DSHA or obtained from the general procedure in Section 11.4. A representative set of acceleration time histories are selected and scaled to be compatible with this outcrop spectrum. Dynamic site response analyses using these acceleration histories as input are used to compute motions at the ground surface. The response spectra of these surface motions are used to define a maximum considered earthquake (MCE) ground motion response spectrum.

The approaches described above have advantages and disadvantages. In many cases, user preference governs the selection, but geotechnical conditions at the site may dictate the use of one approach over the other. On the one hand, if bedrock is at a depth much greater than the extent of the site geotechnical investigations, the direct approach of computing the ground-surface motion in the PSHA/DSHA may be more reasonable. On the other hand, if bedrock is shallow and a large impedance contrast exists between it and the overlying soil (i.e., density times shear-wave velocity of bedrock is much greater than that of the soil), the two-step approach might be more appropriate.

Use of peak ground acceleration as the anchor for a generalized site-dependent response spectrum is discouraged because sufficiently robust ground-motion attenuation relations are available for computing response spectra in western United States and eastern United States tectonic environments.

C21.1 SITE RESPONSE ANALYSIS

C21.1.1 Base Ground Motions. Ground motion acceleration histories that are representative of horizontal rock motions at the site are required as input to the soil model. Where a site-specific ground motion hazard analysis is not performed, the MCE response spectrum for Site Class B (rock) is defined using the general procedure described in Section 11.4.1. If the model is terminated in material of Site Class A, C, or D, the input MCE response spectrum is adjusted in accordance with Section 11.4.3. The United States Geological Survey national seismic hazard mapping project website (http://earthquake.cr.usgs.gov/research/hazmaps/) includes hazard deaggregation options that can be used to evaluate the predominant types of earthquake sources, magnitudes, and distances contributing to the probabilistic ground-motion hazard. Sources of recorded acceleration time histories include the databases of the Consortium of Organizations for Strong Motion Observation Systems (COSMOS) Virtual Data Center website (db.cosmos-eq.org) and the Pacific Earthquake Engineering Research Center (PEER) Strong Motion Data Base website (http://peer.berkeley.edu/products/strong_ground_motion_db.html/), and the United States National Center for Engineering

Strong Motion Data (NCESMD) website (http://www.strongmotioncenter.org). Ground motion acceleration histories at these sites generally were recorded at the ground surface and hence apply for an outcropping condition and should be specified as such in the input to the site response analysis code (see Kwok et al., 2007, for additional details).

C21.1.2 Site Condition Modeling. Modeling criteria are established by site-specific geotechnical investigations that should include: (a) borings with sampling, (b) standard penetration tests (SPTs), cone penetrometer tests (CPTs), and/or other subsurface investigative techniques, and (c) laboratory testing to establish the soil types, properties, and layering. The depth to rock or stiff soil material should be established from these investigations. Investigation should extend to bedrock or, for very deep soil profiles, to material in which the model will be terminated. While it is preferable to measure shear wave velocities in all soil layers, it is also possible to estimate shear wave velocities based on measurements available for similar soils in the local area or through correlations with soil types and properties. A number of such correlations are summarized by Kramer (1996).

Typically, a one-dimensional soil column extending from the ground surface to bedrock is adequate to capture first-order site response characteristics. For very deep soils, the model of the soil columns may extend to very stiff or very dense soils at depth in the column. Two- or three-dimensional models should be considered for critical projects when two- or three-dimensional wave propagation effects may be significant (for example, sloping ground sites). The soil layers in a one-dimensional model are characterized by their total unit weights and shear wave velocities from which low-strain (maximum) shear moduli may be obtained, and by relationships defining the nonlinear shear stress-strain behavior of the soils. The required relationships for analysis are often in the form of curves that describe the variation of soil shear modulus with shear strain (modulus reduction curves) and by curves that describe the variation of soil damping with shear strain (damping curves). In a two- or three-dimensional model, compression wave velocities or moduli or Poisson ratios also are required. In an analysis to estimate the effects of liquefaction on soil site response, the nonlinear soil model also must incorporate the buildup of soil pore water pressures and the consequent reductions of soil stiffness and strength. Typically, modulus reduction curves and damping curves are selected on the basis of published relationships for similar soils (for example, Vucetic and Dobry, 1991; Electric Power Research Institute, 1993; Darendeli, 2001; Menq, 2003; Zhang et al., 2005). Site-specific laboratory dynamic tests on soil samples to establish nonlinear soil characteristics can be considered where published relationships are judged to be inadequate for the types of soils present at the site. Shear and compression wave velocities and associated maximum moduli should be selected based on field tests to determine these parameters or, if such tests are not possible, on published relationships and experience for similar soils in the local area. The uncertainty in the selected maximum shear moduli, modulus reduction and damping curves, and other soil properties should be estimated (see Darendeli, 2001; and Zhang et al., 2008). Consideration of the range of stiffnesses prescribed in Section 12.13.3 (increasing and decreasing by 50 percent) is recommended.

C21.1.3 Site Response Analysis and Computed Results. Analytical methods may be equivalent linear or nonlinear. Frequently used computer programs for one-dimensional analysis include the equivalent linear program SHAKE (Schnabel et al., 1972; Idriss and Sun, 1992) and the nonlinear programs FLAC (Itasca, 2005), DESRA-2 (Lee and Finn, 1978), MARDES (Chang et al., 1991), SUMDES (Li et al., 1992), DMOD_2 (Matasovic, 2006), DEEPSOIL (Hashash and Park, 2001), TESS (Pyke, 2000), and OpenSees (Ragheb, 1994; Parra, 1996; Yang, 2000). If the soil response induces large strains in the soil (such as for high acceleration levels and soft soils), nonlinear programs may be preferable to equivalent linear programs. For analysis of liquefaction effects on site response, computer programs incorporating pore water pressure development (effective stress analyses) should be used (for example, FLAC, DESRA-2, SUMDES, D-MOD, TESS, DEEPSOIL, and OpenSees). Response spectra of output motions at the ground surface are calculated as the ratios of response spectra of ground-surface motions to input outcropping rock motions. Typically, an average of the response spectral ratio curves is obtained and multiplied by the input MCE response spectrum to obtain the MCE ground-surface response spectrum. Alternatively, the results of site-response analyses can be used as part of the PSHA using procedures described by Goulet et al. (2007) and programmed for use in OpenSHA (www.opensha.org; Field et al., 2005). Sensitivity analyses to evaluate effects of soil-property uncertainties should be conducted and considered in developing the final MCE response spectrum.

C21.2 GROUND MOTION HAZARD ANALYSIS

Uncertainties in the characterizations of the key seismic sources (tectonic provinces, zones of seismicity, and active faults), with respect to location, earthquake recurrence, and maximum earthquake magnitude, must be considered in the ground motion hazard analysis. Uncertainties in the ground-motion models are typically included by incorporating more than one ground-motion attenuation equation. However, these equations may underestimate the intermediate- and long-period motion from large earthquakes on nearby active faults due to directivity and directionality effects mentioned in C11.4.7. The probabilistic seismic hazard analysis code can be modified to account for these effects in a consistent probabilistic manner, or a deterministic adjustment can be made to the probabilistic MCE response spectrum using methods in Somerville et al. (1997) and Abrahamson (2000) or more recent procedures. If the deterministic adjustment is used, then judgment must be exercised in selecting the parameters comprising these methods. The worst-case scenario yielding the maximum possible increase in motion from directivity/directionality effects is acknowledged to be conservative, but it offers an upper-bound solution to help gauge the appropriate level for the MCE response spectrum.

Site-response effects in PSHA generally should be evaluated by using the site term in the ground-motion prediction equations. This term is generally a scale factor or a function of V_{s30} = average shear-wave velocity in the upper 30 meters. Site-specific dynamic response analyses can also be performed as described in Section C21.1.

C21.2.1 Probabilistic MCE. PSHA methods are sufficient to define the MCE ground motion at all locations except those near highly active faults. Descriptions of current PSHA methods can be found in McGuire (2004).

C21.2.2 Deterministic MCE. Ground motions for the deterministic MCE shall be based on characteristic earthquakes on all known active faults in a region. The magnitude of a characteristic earthquake on a given fault should be a best estimate of the maximum magnitude capable for that fault but not less than the largest magnitude that has occurred historically on the fault. The maximum magnitude should be estimated considering all seismic-geologic evidence for the fault, including fault length and paleoseismic observations. For faults characterized as having more than a single segment, the potential for rupture of multiple segments in a single earthquake should be considered in assessing the characteristic maximum magnitude for the fault.

For consistency, the same attenuation equations used in the PSHA should be used in the DSHA. Adjustments for directivity/directional effects should also be made, when appropriate. In some cases, ground-motion simulation methods may be appropriate for the estimation of long-period motions at sites in deep sedimentary basins or from great ($M \geq 8$) or giant ($M \geq 9$) earthquakes, for which recorded ground-motion data are lacking.

As a point of clarification, the deterministic lower limit spectrum on the MCE (Figure 21.2-1) extends to zero period in the same manner as the design response spectrum of Figure 11.4-1. The spectrum in Figure 21.2-1 is simply a schematic illustrating the lower bounds for the constant spectral acceleration ($S_{aM} = 1.5F_a$) and constant spectral velocity ($S_{aM} = 0.6F_v/T$) portions of the spectrum. The transition in the deterministic lower limit spectrum from the $1.5F_a$ plateau to zero period occurs at a period (in seconds) of $0.08F_v/F_a$ which is derived in the same manner as T_0 in Section 11.4.5. From this period to zero period, where the ordinate is $0.6F_a$, the deterministic lower limit spectrum is a straight line, similar to the design response spectrum in the period band, 0 to T_0.

C21.3 DESIGN RESPONSE SPECTRUM

Eighty percent of the design response spectrum determined in accordance with Section 11.4.5 was established as the lower limit to prevent the possibility of site-specific studies generating unreasonably low ground motions from potential misapplication of site-specific procedures or misinterpretation or mistakes in the quantification of the basic inputs to these procedures. Even if site-specific studies were correctly performed and resulted in ground-motion response spectra less than the 80 percent lower limit, the uncertainty in the seismic potential and ground-motion attenuation across the United States was recognized in setting this limit. Under these circumstances, the allowance of up to a 20 percent reduction in the design response spectrum based on site-specific studies was considered reasonable.

C21.4 DESIGN ACCELERATION PARAMETERS

The 90 percent lower limit rule, which can affect the determination of S_{DS}, was inserted because it was recognized that site-specific studies could produce response spectra with ordinates at periods greater than 0.2 second that were significantly greater than those at 0.2 second. Similarly, the rule that requires that S_{D1} be taken as the larger of the spectral acceleration at a period of 1 second and two times the spectral acceleration at a period of 2 seconds accounts for the possibility that the assumed $1/T$ proportionality for the constant velocity portion of the design response spectrum begins at periods greater than 1 second or is actually $1/T^n$ (where $n < 1$). Thus, this rule leads to more accurate spectral ordinates at periods around 2 seconds and conservative estimates at shorter periods. However, the conservatism is unlikely to be excessive.

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COMMENTARY TO CHAPTER 22, SEISMIC GROUND MOTION AND LONG-PERIOD TRANSITION MAPS

SEISMIC GROUND MOTION MAPS

ASCE/SEI 7-05 continues to use contour maps of spectral response acceleration (Figures 22-1 through 22-14). The spectral acceleration design maps were prepared by the U.S. Geological Survey (USGS) based on USGS probabilistic maps of the 48 conterminous states (2002), Alaska (1998), Hawaii (1998), and Puerto Rico/Virgin Islands (2003) with modifications based on the 1997 recommendations of the Building Seismic Safety Council. The maps of the 48 states and Puerto Rico/Virgin Islands have been updated from the 2002 edition of the standard but the maps of Alaska, Hawaii, Guam, and Tutuila are unchanged. The USGS also has developed a companion software program that calculates location-specific spectral values based on latitude and longitude or zip code; use of zip codes is discouraged in regions where ground-motion values vary substantially over a short distance. The calculated values are based on the data used to prepare the maps. The spectral values should be adjusted for Site Class effects using the Site Classification Procedure in Section 20 and the site coefficients in Section 11.4. Latitude and longitude for a given address can be found at a variety of websites. The companion software program may be accessed at the USGS website (http://earthquake.usgs.gov/designmaps). The software program should be used to establish spectral values for design because the maps found in ASCE/SEI 7-05 are too small to provide accurate spectral values for many sites.

LONG-PERIOD TRANSITION MAPS

The maps of the long-period transition period, T_L, (Figures 22-15 through 22-20) were introduced in ASCE/SEI 7-05. They were prepared by the USGS in response to BSSC recommendations and subsequently included in the 2003 edition of the *Provisions*. See Section C11.4.5 for a discussion of the technical basis of these maps. The value of T_L obtained from these maps is used in Equation 11.4-7 to determine values of S_a for periods greater than T_L.

The exception in Section 15.7.6.1, regarding the calculation of S_{ac}, the convective response spectral acceleration for tank response, is intended to provide the user the option of computing this acceleration with three different types of site-specific procedures: (a) the procedures in Chapter 21, provided they cover the natural period band containing T_c, the fundamental convective period of the tank-fluid system, (b) ground-motion simulation methods using seismological models, and (c) analysis of representative accelerogram data. Elaboration of these procedures is provided below.

With regard to the first procedure, attenuation equations have been developed for the western United States (Next Generation Attenuation, Power et al., 2006, 2008) and for the central and eastern United States (e.g., Somerville et al., 2001) that cover the period band, 0 to 10 seconds. Thus, for $T_c \leq 10$ seconds, the fundamental convective period range for nearly all storage tanks, these attenuation equations can be used in the same PSHA/DSHA procedures described in Chapter 21 to compute S_a (T_c). The 1.5 factor in Equation 15.7-11, which converts a 5 percent damped spectral acceleration to a 0.5 percent damped value, could then be applied to obtain S_{ac}. Alternatively, this factor could be established by statistical analysis of 0.5 percent damped and 5 percent damped response spectra of accelerograms representative of the ground motion expected at the site.

In some regions of the United States, such as Pacific Northwest and southern Alaska, where subduction-zone earthquakes dominate the ground-motion hazard, attenuation equations for these events only extend to periods between 3 and 5 s, depending on the equation. Thus, for tanks with T_c greater than these periods, other site-specific methods are required.

The second site-specific method to obtain S_a at long periods is simulation through the use of seismological models of fault rupture and wave propagation (Graves and Pitarka, 2004; Hartzell and Heaton, 1983; Hartzell et al., 1999; Liu et al., 2006; Zeng et al., 1994). These models could range from simple seismic source-theory and wave-propagation models, which currently form the basis for many of the attenuation equations used in the central and eastern United States for example, to more complex numerical models that incorporate finite fault rupture for scenario earthquakes and seismic wave propagation through 2-D or 3-D models of the regional geology, which may include basins. These models are particularly attractive for computing long-period ground motions from great earthquakes ($M_w \geq \sim 8$) because ground-motion data are limited for these events. Furthermore, the models are more accurate for predicting longer-period ground motions because: (a) seismographic recordings may be used to calibrate these models and (b) the general nature of the 2-D or 3-D regional geology is typically fairly well resolved at these periods and can be much simpler than would be required for accurate prediction of shorter period motions.

A third site-specific method is the analysis of the response spectra of representative accelerograms that have accurately recorded long-period motions to periods greater than T_c. As T_c increases, the number of qualified records decreases.

However, as digital accelerographs continue to replace analog accelerographs, more recordings with accurate long-period motions will become available. Nevertheless, a number of analog and digital recordings of large and great earthquakes are available that have accurate long-period motions to 8 seconds and beyond. Subsets of these records, representative of the earthquake(s) controlling the ground-motion hazard at a site, can be selected. The 0.5 percent damped response spectra of the records can be scaled using seismic source theory to adjust them to the magnitude and distance of the controlling earthquake. The levels of the scaled response spectra at periods around T_c can be used to determine S_{ac}. If the subset of representative records is limited, then this method should be used in conjunction with the aforementioned simulation methods.

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